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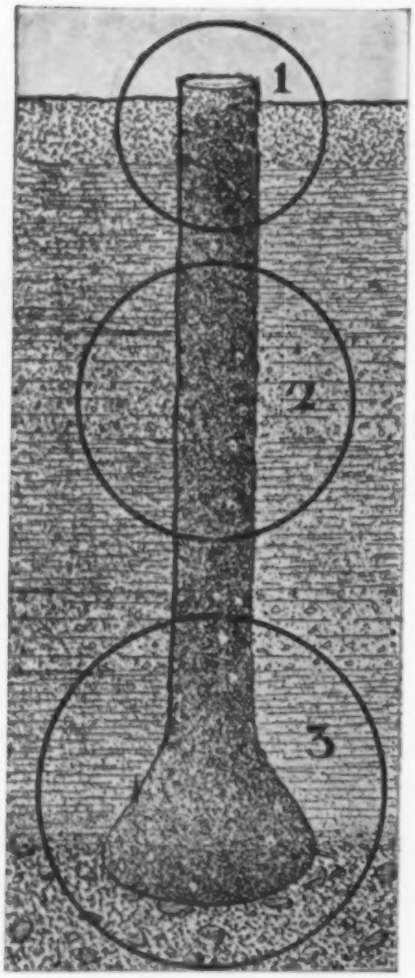
APRIL 1932

Why Piles?

The one fundamental reason for using bearing piles in any foundation work is to transmit the load of the structure through the soft surface soil and distribute it over the firm subsoil.

The Pedestal Pile

[Compressed Concrete]



Theory

Note how ideally the Pedestal Pile meets this fundamental requirement.

(1) At the surface of the ground the Pedestal Pile receives the load on its cylindrical stem—a stem which retains its full diameter throughout its length.

(2) Through the varying strata of soil this shaft transmits the load like a column with surface just rough enough to develop all the friction possible from the surrounding soil.

(3) On the subsoil rests the large spread base distributing the load like a column footing over the firm and compacted subsoil.

Fact

Back of the Pedestal Pile are twenty-two years of successful experience in 37 states in the United States—in Canada—in Japan . . . hundreds of important structures supported by it . . . scores of load tests similar to the following:

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70 tons—1 pile—no settlement

Long Island City, N. Y.
63 tons—1 pile—$\frac{7}{16}$" settlement

New Bedford, Mass.
70 tons—1 pile—$\frac{3}{8}$" settlement

Quebec, Canada
200 tons—4 piles—$\frac{3}{16}$" settlement

Typical large users of Pedestal Piles are:

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Standard Oil Company
Southern Pacific Railway
Canadian Vickers Company
Firestone Tire & Rubber Co.
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North Billerica, Mass. 50 tons—no settlement. Photo shows bulb ends of Pedestal Piles excavated for inspection.

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Among Our Writers

RALPH FREEMAN is senior partner of the firm of Sir Douglas Fox and Partners, of Westminster, London, Consulting Engineers. He received his engineering training at the Imperial College of Science, Kensington, and entered the service of his present firm as an assistant engineer in 1900.

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R. J. PAULETTE has been engaged in municipal and other engineering work in Missouri, Kansas, and Oklahoma for the past twenty years. For the last ten he has been City Engineer of Emporia and Salina, Kans., but has had some private practice in addition. He is now devoting his entire time to private practice.

OREN REED, as a fellow of the American-Scandinavian Foundation, went in 1922–1923 to Norway to study hydro-electric developments. On his return he became Assistant Designing Engineer with the San Joaquin Light and Power Corporation, in Fresno, Calif., on hydro-electric design and construction. In 1929 he received a second fellowship to study the design and manufacture of electrical machinery in Sweden. Since May 1931 he has been Field Engineer for the Indiana Department of Conservation.

THEODORE L. CONDRON has been in private practice as an engineer in Chicago since 1901 and is a member of the firm of Condron and Post, Consulting Engineers. Many of the projects designed by this firm have been of considerable magnitude; they include bridges for railroad companies and municipalities, and buildings for the General Electric Company, the Western Electric Company, the Wagner Electric Corporation, the Ford Motor Company, Sears, Roebuck and Company, and other nationally known concerns. Mr. Condron has contributed a number of papers on building and bridge construction to various publications. He was a Director of the Society from 1923 to 1925.

EARLE R. MATH, after four years of teaching experience in civil engineering at the University of Illinois, became connected with the Western Electric Company in Chicago. After two years with the engineers in the A.E.F. he returned to this company to aid in the preparation of plans and specifications for the architectural and structural work on steel skeleton or reinforced-concrete industrial buildings. Some of these structures have been designed for floor loads as high as 4,000 lb. per sq. ft.

ALFRED D. FLINN, a graduate of the Worcester Polytechnic Institute, received the degree of Sc.D. in 1927 from Louvain University. Prior to his appointment in 1918 as Secretary of the American Engineering Foundation, he had been, successively, engineer of the Metropolitan Water Works, Boston; Managing Editor of *The Engineering Record;* engineer for the Croton Aqueduct Commission; and deputy and deputy chief engineer for the Board of Water Supply of New York City (on the Catskill Aqueduct). Since 1922 he has been Director as well as Secretary of the Foundation. He is the author of numerous technical articles and papers.

ROBERT G. HENNES, after receiving his M.S. from the Massachusetts Institute of Technology in 1928, was employed in the City Engineer's Office, Detroit, in making engineering investigations of materials, construction methods, and design; in supervising core drilling for sewers; and in making condition surveys of older sewers. He is the author of several other published papers on technical subjects.

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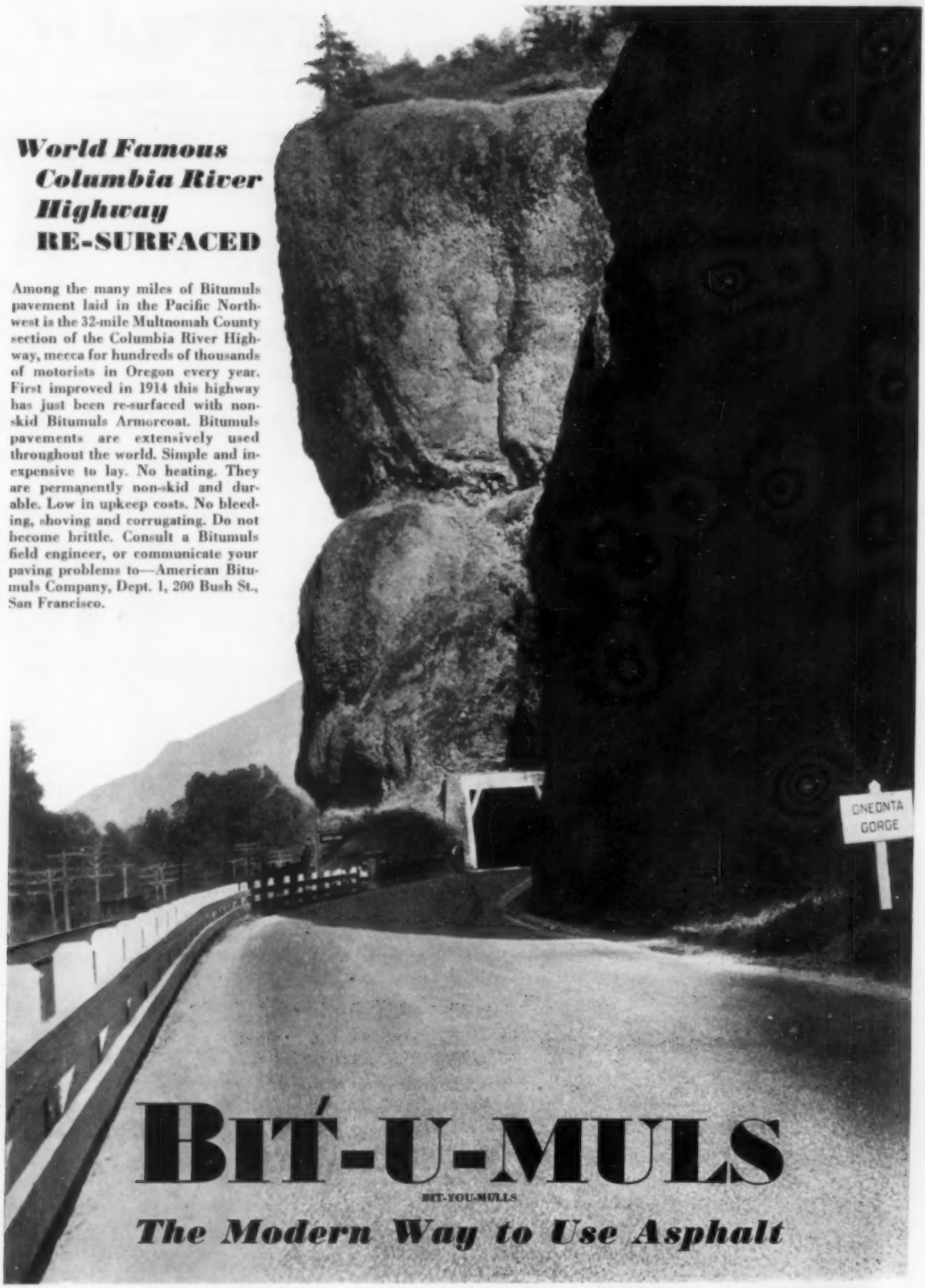

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CIVIL ENGINEERING

APRIL 1932

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VOLUME 2 NUMBER 4

Sydney Harbor Bridge Completed

Main Features of Design, Fabrication, and Erection

By RALPH FREEMAN
MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
SIR DOUGLAS FOX AND PARTNERS, CONSULTING ENGINEERS, WESTMINSTER, LONDON

CONSTRUCTED at a cost of approximately $23,000,000, the 1,650-ft. steel arch across Sydney Harbor, New South Wales, was opened for traffic in March 1932, exactly eight years after the contract was awarded. Bids were submitted by various contractors on alternate types of bridge—suspension, cantilever, cantilever-arch, and arch. The last appeared to be the least expensive, and was adopted. The silicon steel trusses were erected by cantilever methods without falsework. Each of the half arches was tied back by 2⅜-in. cables, 128 in number, looped through curved tunnels dug into the sandstone formation of the approaches. Closure was effected by adjusting the lengths of the cables. By means of a system of 1,000-ton hydraulic jacks operating at the closing point of the arches, each top chord was pre-stressed before the final packings were inserted to form a two-hinged arch. The expensive granite-faced pylons at the approaches are purely a decorative feature, not a structural part of the span.

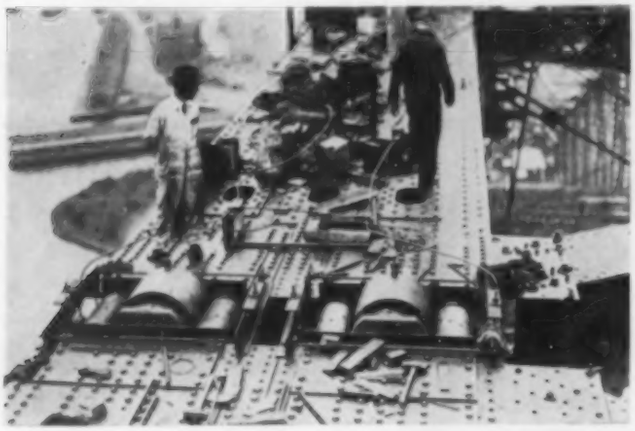
ONE of the most important capitals and trade centers of the British Empire is the city of Sydney, New South Wales, Australia, which has a population of over a million people. It is situated on a magnificient natural harbor, claimed, with much justification, to be the finest in the world. The city has grown chiefly on the south side of the harbor; until the building of the bridge the north side could be reached only by ferries or by a road after a circuitous journey of ten miles. There was no railway service directly connecting the two shores.

Many proposals for bridging the harbor have been made, some of them dating back over one hundred years and most of them suggesting the site that was actually adopted. The bridge is located near the principal business section of the city, at a point where the harbor is very narrow. On account of the high cost of a bridge and the excellence of the ferry service, the actual construction was long delayed.

It was no doubt largely as a result of the efforts of Dr. J. J. C. Bradfield, Chief Engineer of the Public Works Department of New South Wales, that the bridge was constructed. The specifications prepared by him in 1922 defined the essential facilities to be provided by the bridge, the site, the location of the railway tracks and traffic lanes, the live load, the clear span, and the height

SYDNEY HARBOR ARCH BRIDGE, NEW SOUTH WALES
Pylons Separated from Steel Framework. Location of Central Panels Not Obvious

of the harbor crossing. On these specifications bids were invited. The contractors were made responsible for the design of the superstructure and foundations and for the estimates of the quantity of steel, concrete, and excavation required. Bidders were also made responsible for the detailed specifications under which the work was to be carried out.



PRE-STRESSING THE UPPER CHORD WITH HYDRAULIC JACKS

Since the bridge was to have a span 1,650 ft. long and a deck 160 ft. wide, accommodating a 57-ft. roadway, four railroad tracks, and two footways, a structure of unusually massive character was required. Bids were submitted by various contractors for bridges of the suspension, stiffened-suspension, cantilever, cantilever-arch, and arch types. It is interesting to note that although the most valuable experience in building arch bridges has been gained in North America, no American or Canadian firm submitted a bid for this type of bridge, but both of the British bidders did so. As may be seen from the estimates to follow, an arch appears to be by far the least costly type of bridge for the site.

The method adopted for obtaining bids imposed an immense amount of work on the contractors, for they could not know in advance what type of bridge would best meet the preferences of those who were to award the contract. The preparation of the designs required of bidders practically compelled contracting firms to engage the services of consulting engineers. In this capacity the writer was employed by Dorman Long and Company, Ltd., of Middlesbrough, England, and subsequently acted as their adviser throughout the execution of the work.

This firm bid on three types of bridges—cantilever, cantilever-arch, and arch—as shown in Fig. 1. Their estimates, made in 1924, omitting all decorative and other features not structurally essential, were as follows for the spans shown:

Cantilever	1,600-ft. span . . .	$22,200,000
Cantilever-arch . .	1,650-ft. span . . .	18,200,000
Arch	1,650-ft. span . . .	17,100,000

Throughout this article pounds have been converted into dollars at par, £1 = $4.8665.

The lowest bid submitted for a suspension type of bridge was $24,800,000.

Specifications for the arch stipulated massive granite-faced pylons, similar to those of the Hell Gate Bridge. The addition of these pylons increased the bid for the arch bridge to $20,600,000. The actual cost was practically the same as this estimate plus a sum provided for in the contract, representing the difference between the actual wages paid and the amount calculated at the rates ruling at the date of the bid. This extra sum amounted to $2,440,000.

The contract as awarded included the construction of the main span and foundations, the pylons, and the approaches totaling 850 ft. at each end. With unimportant exceptions the contractors carried out the whole construction with their own organization; practically no part of the work was sublet. All the calculations, estimates, designs, working drawings, material and rivet lists, erection plans, and instructions were prepared in London by a special department of the contractor's organization formed for the purpose. All the fabrication, erection, and other work executed in New South Wales was under the personal charge of Lawrence Ennis, a director of the contracting firm.

The trusses of the arch span are designed to carry a live load of 12,000 lb. per lin. ft. plus 10 per cent for impact. The wind load was taken at 30 lb. per sq. ft. acting on the area of the elevation of both trusses, plus $1^1/_2$ times the area of the deck elevation. Provision was made for a temperature range of 60 deg. fahr. For purposes of design, external loads were combined in two ways. In one case full live load was used together with impact, temperature stress, braking load, and one-third the wind load; in the other, three-fourths of the live load was assumed together with impact, temperature stress, braking load, and the full wind load.

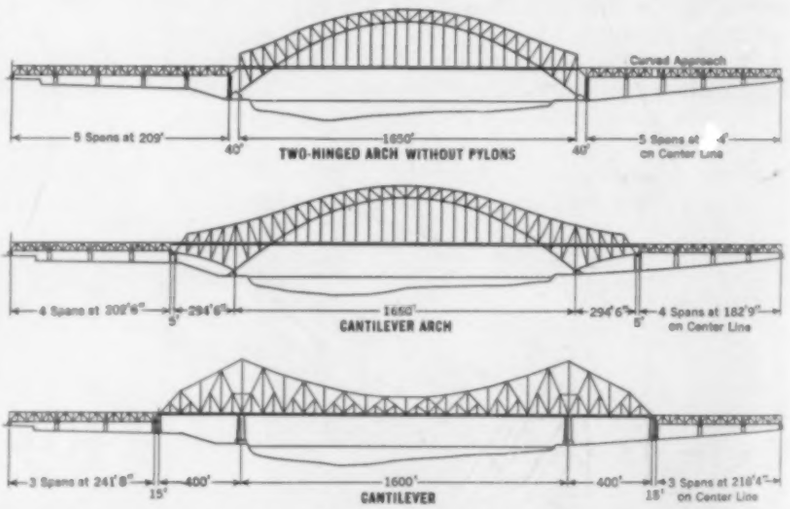


FIG. 1. ALTERNATIVE TYPES OF BRIDGES PROPOSED BY THE SUCCESSFUL BIDDER

The arch trusses were designed using silicon steel with an ultimate tensile strength of from 80,000 to 95,000 lb. per sq. in., an allowable working stress of 23,500 lb. per sq. in. in tension, and 17,500 lb. per sq. in. in compression, with a slenderness ratio, l/r, of less than 50. In the lateral and deck members the specifications required carbon steel having an ultimate tensile strength of from

62,000 to 72,000 lb. per sq. in., an allowable working stress of 18,000 lb. per sq. in. in tension, and of 14,000 lb. per sq. in. in compression, with a slenderness ratio, l/r, of less than 50.

All steel was subjected to bending tests both parallel and transverse to the direction of rolling. The specifications for the manufacture of the steel and for its fabrication and erection were the same as those for the Quebec Bridge, which were published by the Department of Railways and Canals of Canada.

There follows a summary of the weight of the steelwork in the arch span, in short tons:

Trusses	silicon steel	15,550
Cross girders	silicon steel flanges, carbon steel webs	1,450
Bracing	carbon steel	3,500
Hangers	silicon steel	1,300
Stringers, deck, and handrails	carbon steel	5,800
Bearings	carbon steel	1,350
Total weight, in short tons		28,950

STEELWORK MANUFACTURED AT THE BRIDGE SITE

The contractor's organization included no shops or plant capable of constructing steelwork of the exceptional dimensions required, therefore it was necessary to build new shops. It was decided to place these near the bridge site so as to comply with the desire of the government that as far as possible Australian materials and labor be employed, and at the same time to avoid the great cost and risk of delay associated with the shipment of heavy built-up members. The decision to manufacture on the site directly influenced the design of the arch, for it enabled the panel length of the trusses to be fixed at 60 ft., the longest that could be transported in one piece without excessive cost.

Of all the members, the lower chord had the greatest cross sectional area, which reached a maximum of 2,720

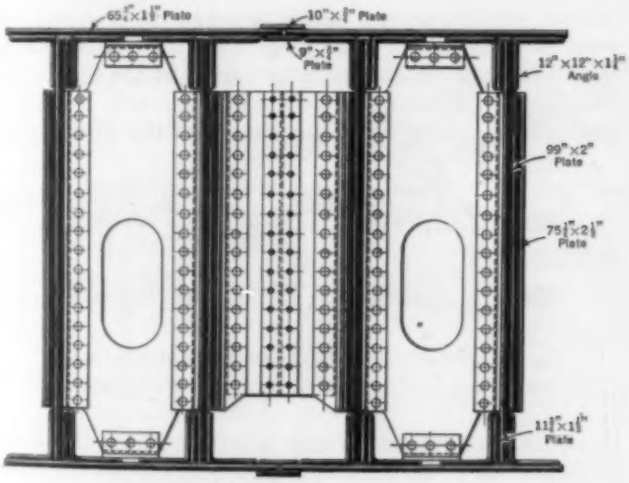


FIG. 2. CROSS SECTION OF A BOTTOM CHORD MEMBER

sq. in. above the bearing. A typical cross section of this chord is shown in Fig. 2. The plates required for its fabrication were of unusual thickness, demanding exceptionally large angles. In the rolling mills the contractor undertook to maintain a standard quality of silicon steel even in plates and sections of these unusual dimensions. The members were designed to be made up of plates up to 8 ft. 3 in. wide by $2^1/_8$ in. thick and weighing up to

CREEPING TRAVELER AFTER ITS RETURN TO END POST
Hangers, Cross Girders, and Deck Erected on the Return Trip

10 tons; and of angles 12 by 12 by $1^1/_4$ in. This exceptionally heavy angle section was found especially valuable in simplifying the design, and it has since been used with advantage in much lighter structures.

In a few special cases the maximum thickness of material to be riveted was as much as 12 in., requiring rivets $1^3/_8$ in. in diameter. The largest rivets used in quantity were exactly $1^1/_4$ in. in diameter and had grips up to 9 in. The holes for such rivets were $1^9/_{32}$ in. in diameter. In making the holes and rivets a high degree of accuracy was maintained. The rivet shanks were of the same diameter throughout. The heads were made conical and closed by a hemispherical tool, and the points were quenched before driving. This procedure resulted in solid filling of the holes.

All abutting members in compression were planed accurately to make close contact over their whole cross section. This was done with an exceptionally massive tool capable of planing an area 12 ft. by 10 ft. When adjacent members were set up in position the ends made perfect contact, so that in members manufactured with the precautions taken for the Sydney Harbor Bridge any excess stress caused by mechanical inaccuracy in the fitting of compression joints would be unimportant.

The specifications stipulated that all butt joints should have splice plates with a cross sectional area equal to that of the member itself and enough rivets to take the whole

stress. This type of splice was therefore used. But it is questionable whether it really would prevent the excess stress caused by inaccurate planing of abutting areas, and whether it would give more rigidity and durability than is secured by the use of an accurately formed butt joint with lighter splice plates.

As illustrated, the bearings for the arch trusses are of unusual design. The pressure on each pin is approximately 45,000,000 lb., and the pin is supported on a forged steel saddle resting on two very heavy web plates. These plates, inclined at an angle of about 27 deg. to the axis, rest on a series of steel box castings, which distribute the pressure on the concrete. The web plates are connected by ten transverse castings. All the connections are made with bolts $4^1/_2$ in. in diameter. In the structural parts the limit of error was 0.001 in., and that in the bolts 0.0001 in.

All the bearings were made at the Darlington Forge, England, where they were completely assembled before shipment. When permanently erected in the bridge, each bearing was supported on four brackets fixed outside the base castings, and resting on hydraulic jacks with crew collars. These jacks were arranged so that they could move a bearing with great exactness in any direction. After the bearings were adjusted to their final position they were still a number of feet above the abutment concrete. Concrete was then rammed into place in the intervening space so as to ensure effective contact with the entire under-surface of the steel casting.

BRIDGE ERECTION INDICATING MAKE-UP OF MEMBERS
Erection Shop in Background

STRUCTURE OF THE ARCH

In preparing the design of the Sydney Harbor Bridge, the precedent of the Hell Gate Bridge and the complete description of it in the PROCEEDINGS of the Society were of great value. The writer desires to acknowledge his indebtedness to the engineers of that structure. In this connection it will be of special interest to note some points of dissimilarity between these two bridges.

In the Sydney Harbor Bridge the shape of the spandrel-braced arch chords is a parabolic curve. The arch is divided into 28 panels, forming, during the erection process, two identical half-arches each of 14 panels. In this respect it differs strikingly from the Hell Gate arch, which has an odd number of panels, necessitating a central panel and a redundant diagonal member. In justification of the central panel the claim has been made that it improves the appearance of the bridge. However, from most angles of vision the central members cannot be easily distinguished. The use of the central panel should be avoided because it requires a redundant member for symmetry. The redundant member complicates the structural problem and the central panel increases the cost of erection.

It is interesting to note that the author of the paper on the Hell Gate Bridge, when he designed the 1,652-ft. Kill van Kull arch (now the Bayonne Bridge) several years later, used an even number of panels. Because of the special method of erection of the Bayonne Bridge, on unsymmetrical temporary piers, there would have been no objection to the introduction of a central panel if this form were preferred.

In an arch bridge the lower chord members near the springing line must be of considerably greater mass than the other members of the structure. There is some difficulty in adopting a form of member which can be varied in weight and at the same time have a uniform breadth so as to permit the placing of connection plates a standard distance apart. In the case of the Hell Gate Bridge, the members are built up with two webs, necessitating lower chord members of great depth in relation to their breadth. In the Sydney arch the total mass of the members permitted the use of four webs, without sacrifice of economy. Thus the surface exposed to transverse wind was reduced. Where the deck passes between the trusses, the moments due to wind must be sustained by the individual lower chord members, and for this function the broad type of member with four webs is advantageous.

In the Hell Gate Bridge no sway bracing was used to connect the two trusses except between the five pairs of posts at each end of the bridge. In the Sydney arch, sway bracing is omitted entirely except at the end posts. The lateral girders between the chords carry the wind load independently of one another, and the whole reaction of the deck lateral system is transferred to the lower chord laterals at the point where the deck crosses the plane of the chords.

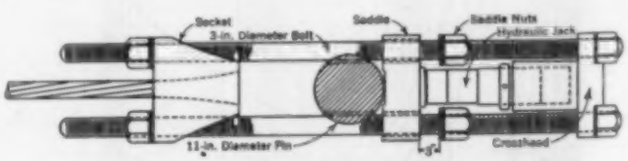


FIG. 3. APPARATUS FOR SLACKENING ANCHOR ROPES
Crosshead and Jack in Position, Saddle Nuts Screwed Back

HALF ARCHES ERECTED AS CANTILEVERS WITH WIRE-ROPE HOLDBACKS

During erection, the trusses were divided into two exactly symmetrical parts, the central post being split vertically in two halves, one attached to each side and

supporting the chord member above it. The junction of the opposed central chord members was made on a pin bearing. At the lower chord, this type of joint is obviously suited to the requirements of erection, as it readily enables the two halves to be lowered together until they meet on the central pin. In the top chord a similar arrangement was adopted, as it was found to be the most satisfactory means of completing the joint to comply with the requirements of the erection process.

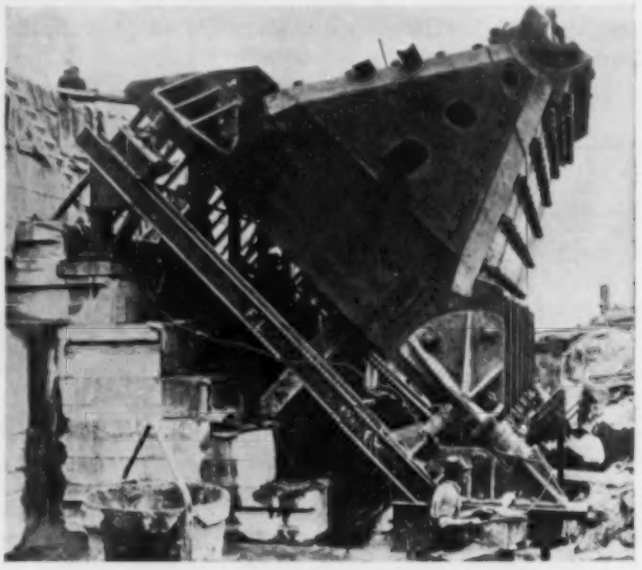
For the erection of the Victoria Falls arch, G. C. Imbault, who was associated with the writer as consulting engineer, used successfully a system of anchorage which was employed on a vastly larger scale for the Sydney Harbor Bridge. Each half of the arch was erected as a cantilever, anchored during this process to the ends of each top chord by 128 wire ropes, each $2^{3}/_{4}$ in. in diameter, and having a breaking strength of about 800,000 lb. The total maximum pull on each anchorage was about 31,000,000 lb. The ropes attached to opposite end posts were really two ends of the same rope, the U-bend of which passed through a tunnel cut into the rock and lined with concrete faced with corrugated steel plates to form a bearing for the ropes, as shown in Fig. 4.

The half arches were so erected that on completion of the cantilevers the ends were about 40 in. apart at mean temperature. To close this gap it was necessary to run out the rope anchorage attachments a distance of 10 in. at each end of the bridge.

Anchor ropes were arranged in 8 tiers of 16 ropes each. Each tier or layer was subdivided into four groups of four ropes, and each set of four ropes was connected by means of saddles to a pin 11 in. in diameter bearing in link plates. These plates were connected to a pin 27 in. in diameter that passed through the end member of the top chord. Individual ropes were attached to the link plates by two bolts passing through a saddle, and these bolts were used to carry out the lowering process.

A temporary crosshead, shown in Fig. 3, was secured to the end of one pair of bolts. By means of a hydraulic jack the tension was transferred to this temporary crosshead, relieving the permanent nuts, which were then screwed back 3 in. The jack ram was allowed to run back until the permanent nuts again made contact. On one wire rope this operation reduced the tension from 240,000 lb. to about 200,000 lb. and increased that of the remaining ropes from 240,000 lb. to about 240,300 lb., causing them to stretch about $^{1}/_{40}$ in. The corresponding movement at the crown of the arch was $^{1}/_{20}$ in., and when made at both ends brought the two sides $^{1}/_{10}$ in. nearer together. The tension on the last rope to be stressed was about 280,000 lb.

This operation had to be repeated three times on all the ropes to complete the closure. As the two halves approached each other, the run of the jacks was increased. Four sets of jacking equipment were used at each end post, enabling the lowering to go on in successive stages so that the process was practically continuous. Work was done in daylight hours only and the whole operation was completed without difficulty in about ten days.



JACKING A MAIN BEARING INTO POSITION
Temporary Jacking Brackets Removed After Placing Concrete

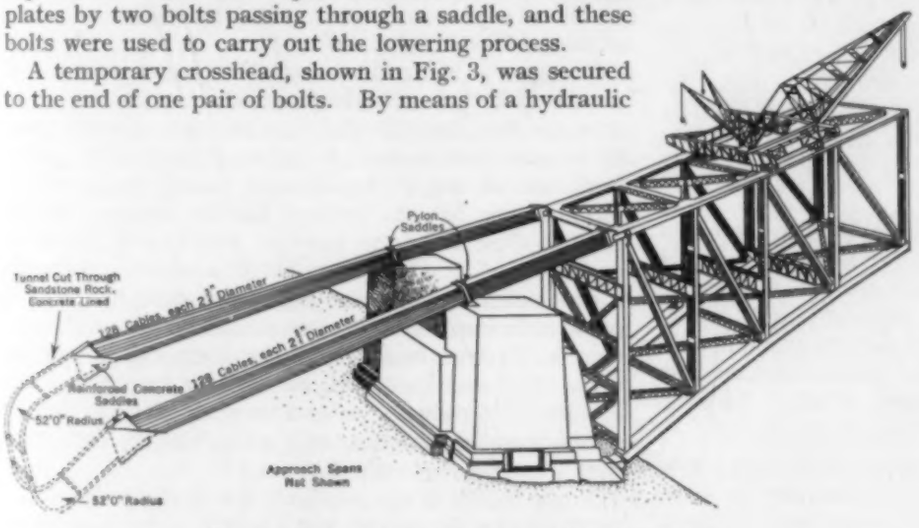


FIG. 4. HALF ARCHES ERECTED AS CANTILEVERS

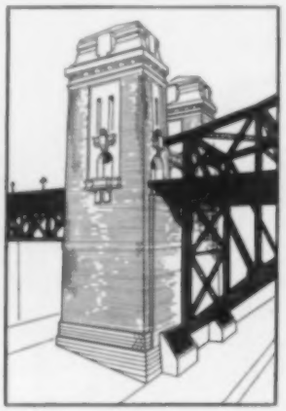
The arches met in excellent lateral alignment. To provide for the possibility of error in alignment and of variation in the relative levels of the two halves, pilot pins 10 in. square were fixed at the extreme ends of the south lower chords to engage the corresponding holes at the ends of the north lower chords. The north half, because of the greater effect of the sun on the top chord and anchorages, moved vertically much more than the south half, and the pilot pins proved valuable.

After the arches met at the lower chords to form a three-hinged arch, the upper members of the two central panels were completed and each top chord was subjected to a compressive stress of about 7,500,000 lb. by means of four hydraulic jacks operating against the flanges of the members. The operation of stressing the top chords was easy to carry out with the excellent plant that was provided. This consisted of eight 1,000-ton jacks, all calibrated in the contractor's testing machine and showing

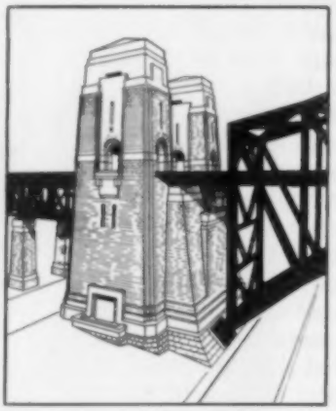
almost incredible uniformity of operation. Each jack was provided with two safety screw jacks to follow up its movement and take the load if necessary.

This method was not used on the Hell Gate Bridge, where the top chords were merely formed under a condition of no stress. The justification for the somewhat costly and elaborate process deliberately adopted for the Sydney Harbor arch by the contracting firm at its own expense would require too long an explanation to be included here. It is my opinion that the cost was justified by decided economies and that structurally the method improves a two-hinged arch.

Hell Gate Arch



Proposed for Sydney Arch



As Built for Sydney Arch

DESIGN OF PYLONS

STEELWORK ERECTED WITH CREEPING TRAVELERS

Steelwork manufactured in the shops was loaded on a lighter and towed to the required position for its erection. This was carried out by two "creeper" cranes, one on each cantilever, designed to handle loads up to 270,000 lb. on the main hoist, 56,000 lb. on a jigger hoist, 11,000 lb. on a walking and slewing jib on the front carriage, and 5,500 lb. on each of two lighter jibs at the back of the main jib.

The lower carriage of the traveler moved on the upper chords of the arch. The upper carriage carrying the main jib traversed the lower carriage. The main jib was raised and lowered by two buttress-thread screws 9 in. in diameter. The panels of the arch chords were built up in two parts longitudinally and joined together after being lifted into place. The hangers, of which the longest was 190 ft., were lifted into place in one piece.

Active erection could not be started for a long period after the contract was let because buildings had to be demolished and the fabricating shops erected and equipped.

The following dates fix the principal events of the erection:

Contract let	March 1924
Steelwork of approach spans commenced	October 1926
Approach spans completed	March 1928
Bearings erected	March 1927 to September 1927
Arch span commenced	October 1928
Arch span closed	September 1930
Deck steelwork of arch span completed	March 1931
Bridge completed	March 1932

The year in which most progress was made in steel erection was 1930, when 44,000,000 lb. was placed.

ESTHETIC VALUE OF PYLONS

At both abutments of the main span are decorative towers or pylons comparable with those of the Hell Gate Bridge. The pylons of the Sydney Harbor Bridge were necessarily of a magnitude proportionate to the size of the bridge, and involved an outlay of over $3,660,000, an amount exceeding the entire cost of the Hell Gate Bridge.

The decision to provide these pylons was made by the government. It is instructive to consider how far so vast an outlay was justified and to trace the development of the form of pylon actually built. These pylons were designed by Sir John Burnet and Partners, a London firm of architects of recognized eminence, engaged by the contracting firm at its own expense in order to ensure as far as possible that the pylons, if built, would do full justice to the structure they were intended to adorn and to the exceptional location. When consulted as to the appearance of the bridge from a purely esthetic standpoint, the architects advised no modification of the form of the arches dictated by the structural requirements. It was their view that the steel structure possessed an inherent beauty of form that justified its erection without pylons.

The pylons of the Hell Gate Bridge are closely connected with the steel structure; the top chords of the arch are actually extended into the pylons and appear to be anchored there. A pylon of somewhat similar form was shown in considerable detail in the official specifications for the Sydney Harbor Bridge. In the opinion of the architects, however, this type of pylon was undesirable. It was thought better to separate the pylon towers definitely from the arch and to employ them to convey the impression of mass stabilizing the steel structure on its true bearings at the springing line of the arch. The architects' original designs emphasized this feature by introducing behind the springing line massive masonry walls, extending only as high as the deck level, with the pylon towers at the rear.

In the pylons as actually built, this wall was reduced to much smaller dimensions, but it is sufficiently pronounced to separate the towers definitely from the steel framework, except at the springing line. The pylon structures are of reinforced concrete faced throughout with granite. If popular opinion can be regarded as a good guide they are a conspicuous artistic success.

In this description many features of interest have been omitted and only scant reference has been made to many others, but I trust that enough information has been included to convey to American readers a useful impression of the structure of the Sydney Harbor Bridge.

Wood Preservation—Economic and Engineering Aspects

By RALPH H. MANN

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FIELD ENGINEER, SERVICE BUREAU, AMERICAN WOOD-PRESERVERS' ASSOCIATION

WHEN timber was plentiful and cheap, there was little incentive for its artificial preservation: but nowadays the need for economy as well as for conservation of natural resources warrants considerable expenditures for treatment to extend its useful life. New preservatives are continually being developed. While results for some of these to date indicate future possibilities, this paper is confined to two preservatives, coal-tar creosote and zinc chloride, both in use for over fifty years. As to the methods used, that of pressure treatment shows the greatest ultimate economy, according to Mr. Mann. How it is effected, what it accomplishes, and where it has been successfully used are indicated in this brief review, which considers the problem from the standpoint of the wood-preservation industry. Although it is not claimed that perfection in treatment has already been attained, it is evident that marked progress has been made.

TODAY wood preservation is a field that offers great possibilities for the solution of problems of engineering design where it is desired to secure economy combined with structural efficiency and widespread utility. The greater percentage of engineering projects require the use of timber. The value of the timber employed where its life is needlessly shortened by preventable decay or insect attack amounts in the aggregate to millions of dollars annually. In the past, preservation by pressure treatment has increased the service life of the timber elements of structures many times at a reasonably low cost.

In the United States, the railroads were pioneers in the use of wood treated for protection against decay. Railway ties were the first wood products to be preserved in quantity, but preservative treatment was soon specified for railway bridge timbers and piling. As service records became available, the efficacy of chemical treatment came to be better appreciated and such treatment was gradually utilized for the protection of the timber used in many different types of construction. Within the last quarter of a century there has been a tremendous expansion in the requirements of American industries for chemically preserved wood.

HISTORICAL STUDY

Early in 1880 the Society instituted a committee to study the "preservation of timber" under the chairmanship of Octave Chanute, later President of the Society. Data gathered from every available source were embodied in the committee's report, presented in 1885. In 1899, Mr. Chanute went abroad for the purpose of investigating practice in the treatment of railway ties in Europe, where timber preservation had been practiced for some sixty years. In his paper entitled "The Preservation of Railway Ties in Europe," in TRANSACTIONS, Vol. 45 (1901), page 498, the difficulty of determining the value of a new preservative is strikingly brought out. "The principal dilemma with reference to new processes is the fact that it takes half a business lifetime (15 to 18 years) to ascertain beyond all peradventure whether an antiseptic or a method is thoroughly efficient to preserve ties in track."

American experiments in the treatment of timber date back to 1856, when the Vermont Central Railway opened a plant for the impregnating of ties and timbers with zinc chloride. The first creosoting plant was built in Massachusetts in 1865. Although later inspection of some of the timber preserved at these plants established the efficacy of chemicals to impart durability, the abundance and cheapness of timber and the relatively high cost of preservation caused an early discontinuance of operation in both cases.

Modern timber preservation in the United States really began in 1875, with the construction of a plant at West Pascagoula, Miss., for the treatment of timber required by the Louisville and Nashville Railway. That same year an earlier plant, at Galveston, Tex., treated piles for the railway bridge across Galveston Bay. This bridge was razed in 1913, and many of the old piles were found in good condition after 38 years in waters infested with marine borers.

A trestle across the eastern end of Lake Pontchartrain in Louisiana, erected in 1883–1884, is still in service. The structural engineer of the Valuation Division of the Interstate Commerce Commission, after an inspection made in 1918, reported the trestle to be in a remarkable state of preservation and estimated that it would have a much longer service life.

In 1904, there were 33 treatment plants operating in the United States. From that time on, the growth of the industry was rapid until, in 1929, there were 156 plants in operation. In 1909 slightly more than 43,000,000 cu. ft. of preserved timber was used, but in 1929 in excess of 362,000,000 cu. ft. was treated.

NATURE OF DECAY

The purposes of wood preservation are three: (1) prevention of decay, (2) protection against insect attack, and (3) protection against marine borers in tidal waters. The greatest damage, of course, results from decay, although the value of the timber in buildings destroyed by insect attack each year in the United States aggregates many millions of dollars.

Decay is caused by fungi, a low, parasitic form of plant life. The spores, which are so minute that they can be seen only with the microscope, are produced by fungi in great numbers and float through the air, often for long distances. In the cracks and crevices of timber they find ideal conditions for germination and soon send forth thread-like filaments, which permeate the wood in all

directions, abstracting food from the wood cells. The wood fiber is thus completely broken down and soon reaches the condition known as "rotten."

Four conditions are necessary for the propagation and growth of fungi: (1) air, (2) moisture, (3) favorable temperature and (4) food. If any of these can be removed, immunity from attack by decay is secured. For example, a pile that is always submerged under water has an insufficient supply of air to permit the growth of fungi; and a piece of furniture, the moisture content of which is always kept below a certain point, will not decay. But in engineering it is usually impracticable to control moisture or temperature, and air can seldom be totally excluded. Poisoning of the food supply, that is, the wood itself, is economically practicable, however, and this is the basic principle of wood preservation. Efforts in late years have been directed largely toward the perfecting of treatment processes. Standard plant equipment has been improved to secure a more rigid control of operation in order to adapt the treatment more closely to the particular use for which the timber is intended.

TEST TRACK FOR CREOSOTED TIES
As Used on the Santa Fe Railroad

EXTENT OF INFECTION

The pioneers in wood preservation really worked in ignorance of the true causes of decay. Although infection by contact had been previously established, Robert Hartig first obtained conclusive proof of spore infection. The cause of decay was not correctly established until 1885, when the results of Hartig's researches were published in Berlin. Then, for the first time, those treating timber had an exact understanding of the problem that must be solved by chemical preservation.

The soil-nesting termite, which is by far the most common of the wood-destroying insects, is active in all but five states of the Union, according to the U.S. Bureau of Entomology. Even these five states are by no means exempt, as cases of damage by termites have been reported from them. Marine borers, notably the teredo and the limnoria, are active along much of our seacoast, particularly the Pacific, the Gulf, and the South Atlantic coast. Even the North Atlantic coast is not immune; invasions of these destructive agents are noted wherever the purification of harbor waters has been accomplished to an appreciable extent. According to service records and the opinion of experts, piling and timber, when pressure-treated with coal-tar creosote, are the least costly structural materials that will resist the attack of marine borers.

PROCESSES OF IMPREGNATION

Whatever the preservative adopted for pressure treatment, two systems of impregnation are in common use, the full-cell and the empty-cell process. Both are accomplished by the same general method. In the full-cell process, a preliminary vacuum is created, the heated preservative is then admitted to the treating cylinders, and temperatures and pressures are gradually raised and maintained until the stipulated amount of preservative has been forced into the timber. The pressure is then released, the preservative drained from the cylinder, and a brief final vacuum is created and maintained until no preservative will drip from the wood when it is removed from the cylinder.

In the empty-cell process, the air in the cylinder is at atmospheric pressure, or else the wood is subjected to an initial charge of compressed air. The preservative is then admitted to the cylinder without altering the pressure. Temperatures and pressures are raised, as in the full-cell process, and maintained until a sufficient quantity of the chemical has been forced into the wood to assure the final retention of the stipulated amount of preservative. Pressure is then released and the preservative drained from the cylinder. The air that was imprisoned during the injection of the preservative expands and, with the help of a final vacuum, forces out the free oil in the wood cells, leaving the cell walls saturated with the antiseptic.

If green timber is to be treated, the actual impregnation of the wood with the preservative must be preceded by preparatory or artificial seasoning. In the case of Southern pine, this is accomplished by steaming the wood in the treating cylinder at a pressure not to exceed

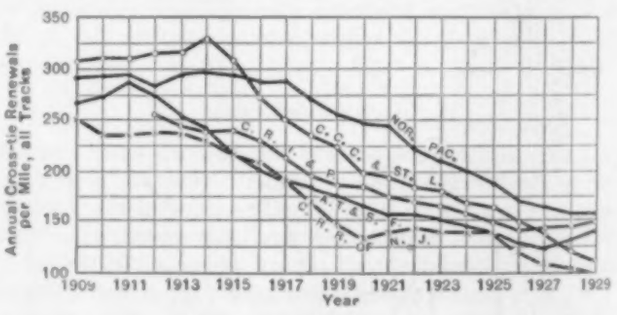


FIG. 1. CROSS-TIE RENEWAL FOR TYPICAL AMERICAN RAILROADS
Points Represent Averages for Preceding Five Years

30 lb. per sq. in. and then subjecting it to a vacuum. Sufficient moisture is thus driven off to prepare the wood for the reception of the preservative. In the treatment of Douglas fir and sometimes of hardwoods, artificial seasoning is accomplished by boiling in the preservative under a vacuum, since high temperatures might tend to damage these woods. During the boiling process the condensate is allowed to escape and the process is continued until sufficient moisture has been evaporated from the wood to make it receptive to treatment.

TYPES OF PRESERVATIVES

Only two preservatives, coal-tar creosote and zinc chloride, will be discussed, as these were used for the preservation of approximately 98 per cent of all the timber treated in 1930, the latest year of record. Both have been in use for well over fifty years, and records are available that establish their efficacy. Creosote is suitable for the preservation of all timber except in places where a very slight odor might be objectionable or where painting is required. Considerable success has been achieved in painting creosoted posts used for highway guard rails with aluminum when an approved paint was used.

Zinc chloride, either alone or mixed with creosote, has been extensively used for the treatment of railway ties, but today it is most widely employed in the treatment of lumber for building construction, and for structures in amusement parks and for other recreational purposes. Lumber treated with zinc chloride is odorless and colorless, and when dry can be painted as effectively as untreated wood.

The amount of preservative that is specified per cubic foot of timber depends largely on the severity of the conditions to be met. For highway bridges in the United States, a 12-lb. retention of creosote is usually specified. Power and telephone companies usually specify for poles an 8, 10, or 12-lb. treatment by an empty-cell process. Formerly railway ties were ordinarily treated with 6 lb. of creosote per cubic foot of wood, where the factor of mechanical wear really determined the life of the tie. With larger tie plates and improved track construction, however, rail cutting and mechanical wear are being materially reduced, and there is a marked tendency to specify heavier treatments. A number of railways are now calling for 8 lb. of creosote per cubic foot of wood. From $^1/_2$ to 1 lb. of zinc chloride per cubic foot of timber is usually specified, although a 1-lb. retention is recommended as more efficient.

OLD CREOSOTED TIMBER-AND-PILE TRESTLE
Has Spanned Lake Pontchartrain, La., for a Half Century

One of the outstanding series of service tests on railroad ties is that carried on for the past 22 years by the Chicago, Burlington, and Quincy Railroad. Reports just submitted by the railroad, including the year 1931, show an estimated service life of ties treated by these two preservatives varying from 24.8 to 14.33 years. In comparison, the estimated life of untreated ties is 5.8 years.

The average annual renewals of cross-ties used by railroads and utilities for certain large railway systems in the United States are shown in Fig. 1, which indicates the general saving in the renewals resulting from treatment. In translating the indicated trend, it must be borne in mind that the true decrease is dependent on the total number of treated ties placed each year, on the size of the ties and tie plates, on traffic conditions, and on the weight of the rail.

DOCK CONSTRUCTION USES CREOSOTED PILING AND TIMBER
Press Street Steamboat Landing at New Orleans

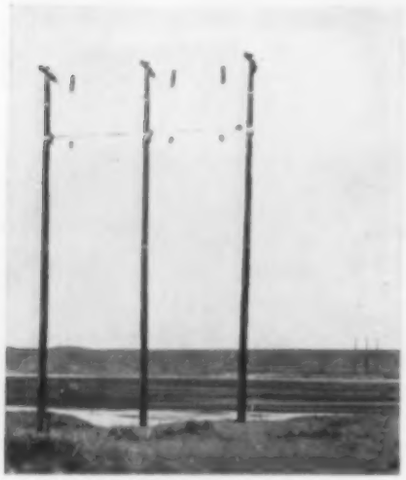
In 1930, C. C. Cook, Maintenance Engineer of the Baltimore and Ohio Railroad and a former president of the American Wood-Preservers' Association, pointed out that American railways were then saving $145,000 per day through the treatment of ties. When an average life of twenty or more years is realized, he stated that "the railroads will save $287,000 per day instead of the present saving of $145,000." On many railroads renewals are practically all made with treated ties, and as these additions raise the percentage of preserved ties in the track an average life well in excess of 20 years may be predicted.

Many creosoted timber bridges that have been in constant service for more than 30 years are still perfectly sound. The Lake Pontchartrain Bridge, now in its forty-ninth year of service, is an outstanding example. Thirty-three such bridges from 26 to 32 years old, which were chosen at random from those of the Santa Fe and Southern Pacific railroads in California, were inspected by a committee of the American Railway Engineering Association during 1928. They were found in a remarkable state of preservation. Not a stringer had been replaced in any of them except in one case where the end of a stringer had been cut after treatment, leaving the untreated heart exposed.

The life of full-length pressure-creosoted Southern pine poles has been variously estimated at from 25 to 40 years. Their use has increased from 67,541 placed in service in 1918, to 2,428,000 placed in service in 1930. This seems to be clear evidence that public utility corporations have come to realize the resulting major economy to be effected through wood preservation.

Among the many classes of construction in which preserved wood is commonly used, the following are outstanding: railroad and highway bridges and trestles, including piling and superstructure; cross-ties; bridge floors; culverts; wood blocks for heavy-duty floors; poles; fence and highway guard-rail posts and timbers;

sills for residences or factories; roof sheathing; mine ties and timbers; oil derrick foundations; grandstand and stadia construction; nailing strips or sleepers under wood floors; sub-floor planking; marine piling and dock timbers; stockyard fencing, barns, and runways; coal tipples; and piling for the support of heavy permanent structures.



POWER LINE CROSSING THE CANADIAN RIVER, TEXAS
Creosoted Southern Pine Poles Support 1,450-Ft. Span

Preservative treatments such as spraying, dipping, and brushing will usually prolong the life of the lumber, but they are at best superficial and the economies resulting from them are not at all comparable with those obtained by the standard pressure processes.

Some of the more recent developments in the field of wood preservation include the following:

1. "Fireproofing," or the treating of timbers so that they will not receive, or, through their own flaming, pass on fire to adjoining combustible materials. For more than thirty years the New York City Building Department has required for certain types of buildings that "fireproof" wood be used.

2. Giving greater consideration to the structural efficiency, permanence, and economy of pressure-treated piling for building foundations.

It has long been the standard engineering practice where wood piles were used in permanent foundation work, to specify that they be cut off below the line of permanent saturation. The mechanical stability and the load-supporting power of wood piles is not questioned by leading engineers, since during many centuries past this type of material has been employed in foundation work. Although certain engineers may not agree that it is the best practice, pressure-creosoted piles are now used in building and structural foundations where they are cut off above the water table. With the pile butts surrounded by earth and the tops encased in the concrete of the footings, these preserved piles are now being considered by many leading foundation engineers as permanent foundation material. Evident economies result from eliminating the cofferdams required for subaqueous work and substituting treated piles in lieu of deep footing masonry.

FOUNDATION PILING FOR A GRAIN ELEVATOR OF 13-MILLION-BUSHEL CAPACITY, AT ALBANY, N.Y.
Cut-Off Elevation 10 Ft. Above Permanent Water Table

ECONOMICS OF PRESERVATION

The relative costs of treated and untreated timber may best be compared on the basis of annual charges for the material in place. The usual equation for computing the annual charge is:

$$A = Pr\left(\frac{(1+r)^n}{(1+r)^n-1}\right) \quad . \quad . \quad . \quad . \quad . \quad [1]$$

in which: A = the annual charge
P = first cost
n = number of years of service life
r = rate of interest expressed decimally.

When the service life of railroad ties, treated and untreated, can be predicted, the economy derived from treatment can be readily computed. For example, assume that:

Rate of interest	6 per cent
Untreated tie:	
Cost installed in track	$1.40
Estimated life	6 years
Treated tie:	
Cost installed in track	$1.80
Estimated life	23 years

Using Equation [1], the annual charges will be found to be:

For the untreated tie, costing $1.40 (6-year period)	$0.284
For the treated tie, costing $1.80 (23-year period)	$0.146
Annual saving in favor of treated tie	$0.138

Assuming that there are three thousand ties per mile

of track, the annual saving obtained by the use of treated instead of untreated ties will be $414, a substantial amount indeed when applied over the many miles of a railway in operation.

It is frequently desirable to ascertain the expense justified for treatment. In a paper read before the American Wood-Preservers' Association in 1925, J. D. MacLean, Engineer, U.S. Forest Products Laboratory, Madison, Wis., suggested the formula:

$$100\,\frac{A_u - A_t}{A_t} \quad \ldots\ldots\ldots \quad [2]$$

for use with Equation 1 in determining the percentage by which the first cost of untreated timber might be increased to secure longer periods of service life. In Equation 2:

A_u = unit annual charge on untreated timber

A_t = unit annual charge on treated timber

For example, let it be required to find the percentage of increase in cost justified for treatment where a life of 5 years may be expected for the untreated material and a life of 25 years for the treated material. By Equation 1, the annual charge for ties having a service life of 5 years is $0.237; and that for ties having a life of 25 years is $0.078. Then, by Equation 2,

$$\left(\frac{0.237 - 0.078}{0.078}\right) 100 = 204 \text{ per cent}$$

The same formula is useful in determining the increase in cost justified to secure longer-lived structures generally. For example, with the proper substitution, it may be shown that the cost of a bridge having a service life of 30 years need be increased by only 21 per cent to secure a structure with 100 years of service, if equal ultimate economy is to be realized.

The increase in life which treatment must secure to justify its cost is computed by dividing the annual charge for the untreated product by the cost of the treated. Thus, assume that an untreated railway tie costs $1.00 and will last 5 years, while the same tie treated costs $1.50. Its annual cost untreated, based on a 5-year life, is $0.237.

Therefore, $\dfrac{0.237}{\$1.50} = \$0.158$

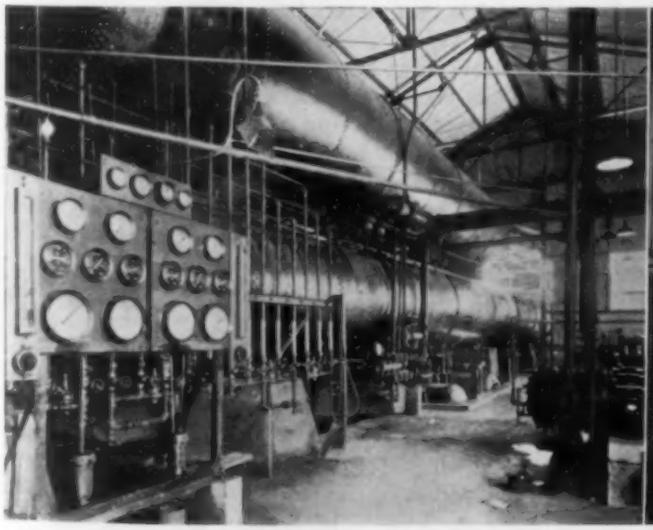
Again referring to Equation 1, it is found that $0.158 is the annual charge for between 8 and 9 years; therefore

PRESERVED WOOD FOR A BOARDWALK
Construction Treated with Zinc Chloride
at East Kansas City, Mo.

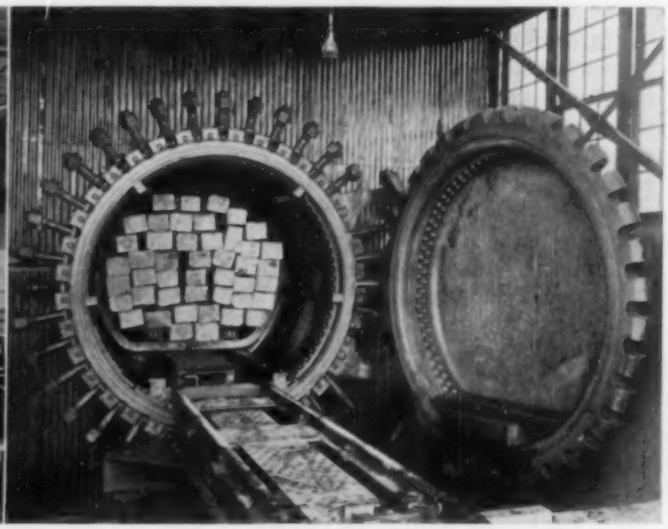
if treatment at this cost increases the service life of the installation by only 4 years, the treated product is as economical as the untreated. Any increase beyond this period of course results in clear gain.

If the maximum possible economy through treatment is to be realized, consideration must be given to the selection of proper grades of lumber for treatment, particularly for those structures in which structural grades of timber are required. Where the wood is to be used in its untreated state, heart content is a desirable qualification for durability. This, however, increases cost and the presence of a high percentage of heartwood detracts from the ease with which the wood takes a preservative treatment.

Since durability is imparted by the preservative, these high grades are unnecessary and operate against economy and good treatment. Both the Southern Pine Association and the West Coast Lumbermen's Association



Control Equipment, Treating Cylinder, and Preservative Tank



Open End of a Pressure Cylinder

MODERN WOOD PRESERVING PLANT

recommend suitable grades for treatment. In structures where treated lumber is to be used, reference should be made to these recommendations, as they specify the grades that experience has proved to be the most economical.

Relative to the mechanical properties of heartwood and sapwood, the following paragraph is quoted from *Technical Notes* (189) of the Forest Products Laboratory, U.S. Forest Service, Madison, Wis.

SULPHUR LOADING DOCK AT FREEPORT, TEX.
Wood Preserved with Zinc Chloride Shows No Decay After 12 Years of Service

"In over 500,000 tests on the various species of wood grown in the United States, no effect upon the mechanical properties of the wood due to its change from sapwood into heartwood has ever been noticed. The heartwood of oak, pine, and other structural timbers is not intrinsically stronger than the sapwood, as has often been supposed to be the case; nor is the sapwood of hickory and ash intrinsically stronger than the heartwood, as is sometimes claimed in connection with handle stock."

The Manual of the American Railway Engineering Association, under the subject of "Wooden Bridges and Trestles" (Committee VII) specifies that:

"Piles for use with preservative treatment shall have no sapwood restrictions but have, preferably, as much sapwood as possible."

Similarly, with regard to its coded specifications for structural grades, the phrase is used throughout: "Sapwood wanted for treatment."

To secure durability, field cuts should of course be reduced to the absolute minimum necessary for the erection of the structure. Railroads have given particular attention to this problem and have reduced the necessary field cuts practically to the cutting off of piling to grade. Care should be exercised in handling or storing treated material to avoid breaking the surfaces. It is often properly specified that cant-hooks, dogs, or pike poles be not used with treated piling. Rope slings are recommended for handling all treated material.

One point to be especially noted is the manner of protecting the cut ends of creosoted piles. It is important that two or three brush coats of hot creosote oil be applied to such ends, after which, especially for trestle work, a coat of hot pitch should be added. Then, before the placement of the cap timber, many railroad and port engineers require that a layer of galvanized iron, tar paper, or felt be placed over the pitch. The overlapping corners of such material are then nailed to the body of the pile.

The modern timber-treatment plant is equipped to do even elaborate framing before treatment, and prefabrication of the material should be specified. Intricate patterns required for bascule bridge floors are cut to size and bored before treatment. There is no trouble in securing proper fit in the field, since ordinarily a tolerance of only $^1/_{16}$ in. is permitted between the plan and the dimensions of the treated piece.

EFFORTS CONTINUING

In this article have been briefly summarized some of the actual accomplishments of wood-preservation engineers in their efforts to overcome what has been considered the chief drawback to the use of wood for many construction purposes—its susceptibility to decay and insect attack. These accomplishments are the result of much careful study of wood, of preservative materials, and of methods of treatment. The records of service cited have been obtained from wood treated with preservatives that are considered standard by the wood-preserving and wood-consuming industries. It is the firm conviction of those engaged in timber treatment that careful application of the information gained by these studies, and adherence to the specifications of the American Wood-Preservers' Association for preservative treatment, will insure a durable construction material.

One further aspect of wood preservation deserves emphasis. The Federal Government has shown a decided interest in the preservative treatment of timber, in line with its policy of conserving national resources. When President Hoover held the portfolio of Secretary of Commerce, he instituted a research committee known as the National Committee on Wood Utilization, which went thoroughly into the subject of wood preservation. The committee is still functioning and has produced a number of excellent treatises on this subject. Conservation is accomplished when, through preservative treatment, a pole, for example, may be made to last three times as long as it would in its natural state. It is evident that in this case two trees are saved.

In 1930, the committee published the treatise, *Treated Lumber, Its Uses and Economies*, which is an excellent practical exposition of the subject of wood preservation in the building field. The Society can pride itself on sharing in this valuable work through continuous representation on this committee.

CHANNEL CONDITION OF SMOKY HILL RIVER, LOCATION A
The High-Water Mark Shown Was 10.5 Ft. Above the Water Level Pictured

MINOR DRIFT PARTIALLY OBSTRUCTED SOME REACHES OF THE RIVER
Condition at Location B

Measuring the Discharge of the Smoky Hill River

Hydraulic Characteristics Determined at Flood Stage

By R. J. PAULETTE
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IN TECHNICAL literature there appears to be a shortage of definite information as to the value that should be given the roughness coefficient for rivers in flood. This article by Mr. Paulette is a complete description, in both picture and diagrammatic form, of the regimen of a four-mile reach of the Smoky Hill River. The engineer engaged in the study of river hydraulics will find in it valuable information to aid him in estimating the roughness coefficient for similar river conditions elsewhere.

IN CONNECTION with a rather extensive flood control project undertaken by the City of Salina, Kans., accurate determinations of the roughness coefficient of the Smoky Hill River at bank-full stage were made. The data obtained not only give a roughness coefficient for a stream of fairly well defined physical characteristics but also illustrate the effect of drift on stream flow.

Above Salina, the Smoky Hill River, Fig. 1, has a drainage area of 8,300 sq. miles, and a valley about 300 miles long, which varies in width from a maximum of 45 miles near the upper end, to a minimum of 2 miles just above Salina. The river rises in the eastern part of Colorado and flows east across Kansas until it encounters, about 20 miles south of Salina, the ridge of shale hills that turns it practically due north. This stretch of the river is extremely crooked and traverses a narrow watershed without inflow. Shale is exposed in spots along the right bank, but in general the river bed is in heavy alluvial silt underlaid by sand. The banks are steep and bordered by trees and underbrush. The cross section is fairly uniform.

The flow measurements here recorded were taken along four miles of the river near Salina, where the channel is less tortuous than usual. At ordinary low water the stream is from 50 to 70 ft. wide and has steep banks of

TABLE I. STREAM FLOW DATA FOR THE SMOKY HILL RIVER NEAR SALINA
At 2:40 p.m. May 8, 1930

River Cross Section	Distance Above Crawford Ave. Road In Ft.	Distance Between Sections In Ft.	Water Elevation In Ft.	Fall Between Sections In Ft.	Slope	Area In Sq. Ft.	Average Area In Sq. Ft.	Values				
								p	r	v	c	n
Crawford Street Bridge	0		23.66			1,272		125				
		1,800		0.41	0.000228		1,192	115	10.4	2.43	50	0.050
No. 2	1,800		24.07			1,111		105				
		3,000		1.08	0.000360		1,143	125	9.1	2.53	44	0.055
No. 3	4,800		25.15			1,175		145				
		1,650		0.56	0.000339		1,198	132	9.1	2.42	44	0.055
No. 4	6,450		25.71			1,220		120				
		1,350		0.31	0.000230		1,332	148	9.0	2.17	48	0.049
No. 5	7,800		26.02			1,445		175				
		3,200		1.70	0.000531		1,386	165	8.4	2.09	31	0.078
No. 6	11,000		27.72			1,327		155				
		6,300		2.48	0.000394		1,345	145	9.3	2.15	36	0.068
No. 7	17,300		30.20			1,363		135				
		3,900		0.98	0.000251		1,358	142	9.6	2.13	43	0.057
Upper Bridge	21,200		31.18			1,352		150				
Total	....	21,200	....	7.52	.....	....	....	...	...	...	..	....
Average	....	....	....	...	0.000355	1,283	....	139	9.2	2.26	40	0.060

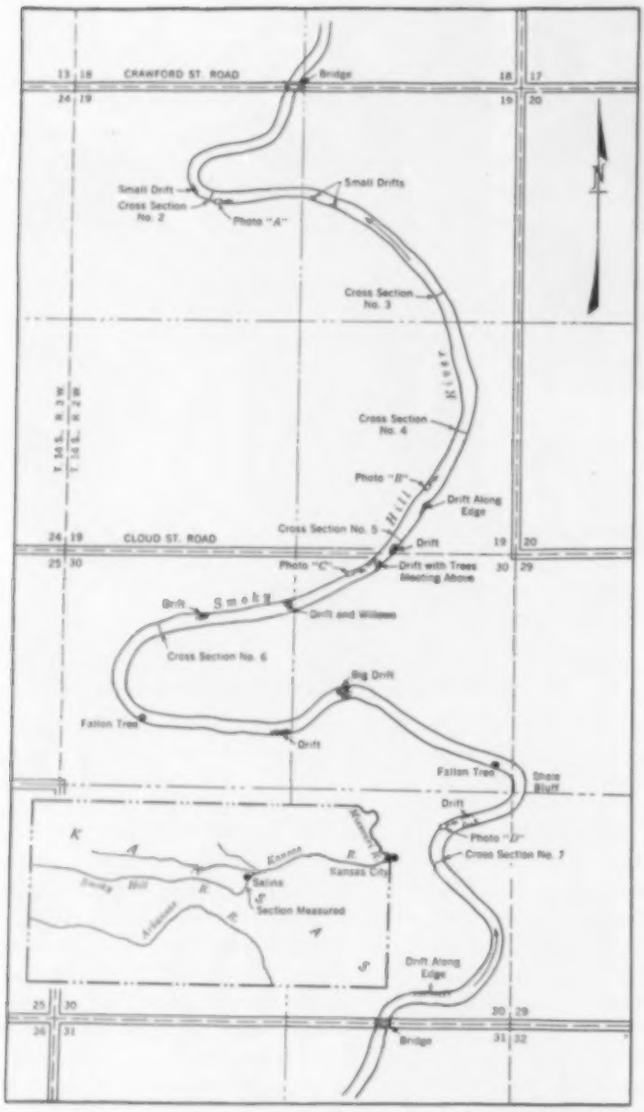
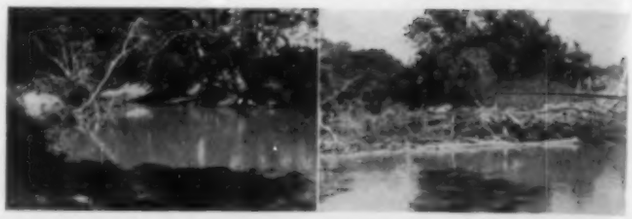


FIG. 1. SMOKY HILL RIVER SHOWING LOCATION OF MEASUREMENTS TAKEN SOUTH OF SALINA, KANS.



THE RIVER FROM LOCATION C—TREES TIPPED BY SLIDING BANKS

AT LOCATION D, DRIFT COLLECTED ON BOTH BANKS

gumbo. They are lined with trees from the water line for about 50 ft. back, but the underbrush is not heavy. In the upper part of the measured section, south of cross section No. 5, the channel is obstructed by drifts and bank slides carrying trees and underbrush. The lower part, north of cross section No. 5, is fairly clean. A topographic map of the area, to a scale of 200 ft. to the mile, and accurate river cross sections had been completed just prior to the high water of May 8, 1930. On that date the crest of high water, due to rains several days earlier, reached the stretch of river shown in Fig. 1. The measurements were recorded near the end of the rise.

Discharge measurements were taken at two highway bridges two miles apart. Velocities were measured with a current meter, and discharges were corrected for the angle of approach. At the beginning and end of the measurement period the water level was recorded at each of the six river cross sections between the two bridges. All readings were timed and corrected to a mean time, 2:40 p.m. The adjusted discharges checked within 3 cu. ft., and the water levels recorded for intermediate stations were all consistent.

Although the cross sections were not re-measured, a careful inspection after the flood of the entire reach of the river showed no evidence of a change in conditions which would indicate any scouring action during high water. The high-water velocity of from 2.0 to 2.5 ft. per sec. was not greatly above that often assumed for normal flow. In general, the soil is heavy and there is no sand in the river bed. Section No. 3 is in shale.

In Table I are shown the slopes, areas, wetted perimeters, hydraulic radii, velocities, and values of n and c. The resulting discharge measurements are as follows:

DISCHARGE DATA	CRAWFORD BRIDGE	UPPER BRIDGE
Elevation when measured	23.61 ft.	31.25 ft.
Discharge measurement	2,880 sec-ft.	2,912 sec-ft.
Mean elevation (2:40 p.m.)	23.66 ft.	31.18 ft.
Adjusted discharge	2,895 sec-ft.	2,892 sec-ft.

The character of the river channel is shown by the accompanying figures and photographs. On the map of the river are indicated the location of the bridges, the river cross sections, the drifts, and the points at which photographs were taken. The various river cross sections shown in Fig. 2 were plotted with the mean water level as a datum. The photographs were taken at a river stage about 10.5 ft. below that at which the data were obtained, but the high-water mark is indicated on them.

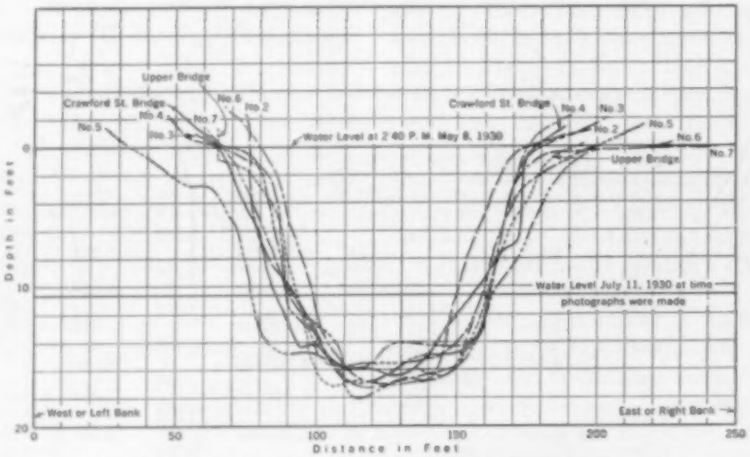


FIG. 2. CROSS SECTIONS OF SMOKY HILL RIVER IN FLOOD
Mean High Water Taken as Datum
(Time Adjusted to 2:40 p.m.)

Acknowledgment is made of the assistance of W. C. Wiggin, assistant engineer in charge of the field work for the flood control studies carried on south of Salina during the time I was City Engineer of the city.

Disintegration of Dams in Scandinavia

A Survey of Some Important Concrete Structures

By Oren Reed

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During the past thirty years many concrete dams have been built in Norway and Sweden to make use of the vast natural water resources of those countries. An examination of 25 of these dams by Mr. Reed revealed many important structural deficiencies. The serious effects of percolating water, frost action, ice abrasion on upstream faces, and erosion of spillway aprons by ice blocks and logs, have shown the necessity for improved construction methods. The remedial measures resorted to have included pressure grouting, use of waterproof coatings, and construction of a separate concrete water face. Badly eroded spillways have been rebuilt of timber-clad concrete mats anchored to the rock of the river bed, the timber being protected with strap iron. Mr. Reed throws much light on the problem of designing and constructing concrete dams to withstand these deteriorating agencies. His conclusions should be helpful to all engineers concerned with such problems.

THE oldest important concrete dams in Norway and Sweden were constructed about thirty years ago. Before that time dams in those countries were built of wood, with an earth fill; or of masonry, constructed either as a dry wall or laid up in cement mortar. The building of large concrete dams for storage and regulation was rapid after 1905. The value of these structures is considerable and the investments in power houses, industrial plants, farms, and homes that would be endangered by their failure are much more. The safety of these dams is therefore of the greatest importance. Hydraulic engineers have known for many years that concrete made from ordinary portland cement is not very durable when subjected to water pressure on but one side. Dams built in recent years incorporate the results of tests and investigations on concrete and construction methods, and are more lasting than the older structures.

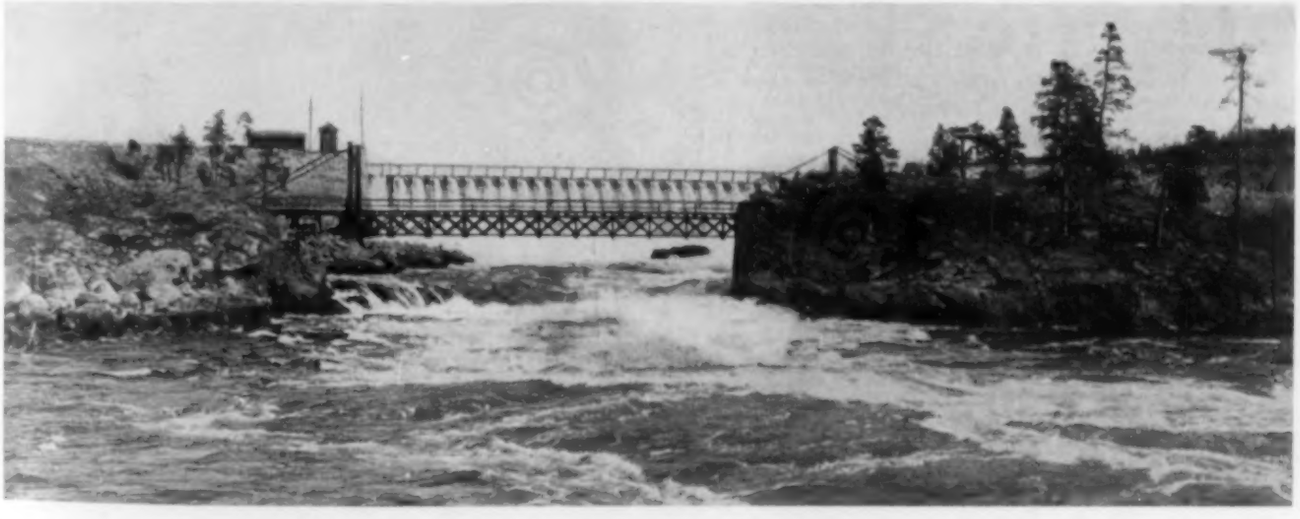
In Nature's laboratory, tests on full-sized models are carried to completion in a relatively short time. Concrete dams in Norway and Sweden have been subjected to severe climatic conditions, which have hastened the destructive action of other agents. Previous ideas concerning the permanence of concrete have been modified in late years because of the corrosion of large and important dams. Some structures have developed considerable leakage and others have shown heavy coatings of calcium carbonate and flaking or spalling of the concrete at the surface. Fundamentally, most of the deterioration of concrete has been due to its being porous.

During the course of this investigation I made two trips to Scandinavia—one in 1922–1923 and one in 1929–1930—in the course of which I visited 14 storage and diversion dams in Norway and 11 in Sweden, as listed in Table I.

From these dams I have chosen for detailed description four of the largest and most important, so as to give a general picture of such concrete structures in Norway and Sweden.

The body of the Tunhövd Dam was constructed between 1916 and 1920 of a 1 : 3 : 5 concrete mix, its up-stream face being protected with a rich layer of 1 : $2^1/_4$ mortar from 12 to 20 in. thick. There is a granite facing on the downstream face, with 10 to 12-in. courses. The overfall surface of the ogee spillway also has a granite facing with 12-in. courses. A system of tile drainage pipes was installed 59 in. from the water face of the dam.

The concrete was transported in two-wheeled hand carts and deposited in 10-in. layers. Concrete forms were thoroughly vibrated by means of compressed-air



Thirty Miles Above the Arctic Circle
The Porjus Dam Regulates the Lule River, Sweden

hammers. After the concrete of the body of the dam was partly cured and a rich mortar layer was placed on the water face, its troweled surface was painted with waterproof paint. The average 28-day compressive strength of the normal mix was 2,880 lb. per sq. in.

TUNHÖVD DAM, NUMEDALSLAAGEN RIVER, NORWAY
Entrance to Logging Chute in Foreground

The high-water level in the reservoir is at elevation 2,414 ft. above sea level. Draft on the reservoir begins in the autumn so that the water level is lowest at the beginning of the spring floods. The mortar of the water face is investigated each spring, when the reservoir is at its lowest, for cracks and flaking. Each year it is found that there is peeling or flaking of the surface concrete. A loose, brown, sandy mass is usually found under each flake or disintegrated section. Brownish veins lead from such disintegrated spots to a depth of from 14 to 16 in. Disintegrated parts are removed by air-hammers and hand-chisels and replaced with rich mortar and gunite. On the spillway, the joint mortar of the granite facing must be periodically renewed. The mortar back of the facing has also been found to have the same sandy composition and brown color as that on the water face. On the other hand, there is no indication of disintegration of the concrete in the inspection gallery of the dam. The total leakage in the inspection gallery has been nearly constant from year to year.

TABLE I. STORAGE AND DIVERSION DAMS IN NORWAY AND SWEDEN

PLANT	RIVER
Norway:	
Kykkelsrud	Glomma
Vamma	Glomma
Mörkfoss-Solbergfoss	Glomma
A/S Borregaard	Glomma
A/S Hafslund	Glomma
Raanaasfoss	Glomma
Tunhövd, Nore	Numedalslaagen
Tinsjö Regulation Dam	Tinnelva
Aarlifoss	Tinnelva
Svaelgfoss	Tinnelva
Lienfoss	Tinnelva
Tinnfoss	Tinnelva
Ringedalsvand	Tysse
Lerfoss	Nidelven
Sweden:	
Trollhätten	Göta
Lilla Edet	Göta
Porjus	Lule
Norrfors-Intake	Ume
Norrfors-Regulation	Ume
Hammerforsen	Indal
Älvkarleby	Dal
Lanforsen	Dal
Untra	Dal
Forshuvudforsen	Dal
Bullerforsen	Dal

On the water side of the dam, disintegration seems to have begun near construction and contraction joints. Wherever the bad concrete was found between two construction courses, the lower course was not damaged greatly, but the bottom of the upper course showed considerable disintegration, which had advanced inward and upward from the face. The observed damage occurred at points where probably water had had an opportunity to percolate into the concrete. Water had been able to enter through hair cracks in the mortar facing and had seeped in at the weakest points, the construction joints. Because of the action of the water, the rich facing concrete was changed to a light-colored mass, which was extremely porous. In the next stage, the concrete was cut up by brown veins; later the whole mass became brown and consisted of brown grains and small bits of the good concrete which were loosely held together.

Such damage spread in all directions—except downward into the lower construction course—even to the mortar facing, which was eaten up from the inside. The troweled facing was found to be more resistant than the poured facing. A loose, brown, disintegrated spot might be discovered back of a seemingly undamaged surface. Similar phenemona have been noted back of the granite facing of the spillway.

Corrosion and disintegration were caused by the pressure and composition of the water and by the action of freezing. The water at the Tunhövd Dam is very pure and soft; it is practically free from lime. It has therefore a certain dissolving capacity for the calcium hydrates of the concrete. The water also contains some humic acid, which is not neutralized. Even if the degree of acidity is not great, it would have an effect in the course of time where there is a steady passage of water through the concrete.

RINGEDALSVAND DAM, TYSSE RIVER, NORWAY
Construction Cranes Placing Face Slab

During the winter season the water level is gradually lowered through a range of about 60 ft. During this time the temperature varies between +18 and −40 deg. fahr. The spillway is closed by needles but leakage through the timbers keeps the over-fall surface wet and ice-covered during the early part of the winter. As the water level in the reservoir sinks, the ice cover, which may be over 30 in. thick, is broken up along the face of the dam, and has caused considerable abrasion. After

the water level has sunk, the water absorbed by the facing mortar is frozen, causing hair cracks and spalling. No damage has been noted in the concrete below the normal low-water level.

WATER-TIGHT VERTICAL DECK ADDED TO RINGEDALSVAND DAM

The first part of the Ringedalsvand Dam, built in 1912–1913, was constructed of granite blocks pointed up with 1 : 2 mortar. This section had a maximum height of 54 ft. From 1914 to 1918 the dam was raised by constructing a concrete structure back of the initial section and incorporated with it to give a maximum height of 112 ft. The body of the dam is constructed of 1 : 5 : 6 concrete with about 30 per cent plums. There is a granite facing on both upstream and downstream slopes. Back of the stone facing on the water side there is a tightening layer of 1 : $2^1/_2$: 3 concrete. The thickness of this rich concrete, measured from the water face, varies from 3 ft. at the crown to 10 ft. at the top of the original masonry dam. Immediately behind the rich tightening concrete is a drainage system consisting of a double row of vertical drain pipes on 24-in. centers. These vertical pipes are connected to a horizontal header, which empties into the inspection gallery, whence the water is conducted to the downstream face of the dam.

Sand and gravel for mortar and concrete were crushed from granite. Construction was started early each spring, usually in April, and continued until late in November. Above the top of the cyclopean section the dam is divided by contraction joints into blocks from 62 to 115 ft. in length.

The water level in the reservoir varies greatly. The reservoir is usually full in July; release starts about October; and the water level is at the foot of the dam in March or April. The minimum temperature at the dam is from −4 to −13 deg. fahr. The crown elevation of the structure is 1,526 ft. above sea level.

EAST SPILLWAY SECTION, PORJUS DAM, LULE RIVER, SWEDEN
Earth Fill with Two Multiple-Arch Spillway Sections

Systematic measurements of the leakage through the dam were started in 1925. The measured leakage in 1928 amounted to about 8,000 acre-ft. The difference between the amount of solid matter in the lake water and that in the leakage water showed that about 30 tons of solid matter were washed out of the dam each year. Since the mortar in the facing joints was not seriously damaged, most of the solid matter came from the tightening layer and the lean concrete of the body of the dam.

Diamond drill cores indicated that where the pouring of concrete had been continued on a surface that had hardened, no care had been taken to obtain a good junction of the old and new concrete. A definite crack remained and as a rule the concrete immediately above the crack was porous. The remaining part of the pour was usually good concrete, having an average compressive strength of 7,100 lb. per sq. in. (the concrete was from 8 to 12 years old). However, there was a great variation in the compressive strength of the test pieces.

STAGES IN CONSTRUCTION OF FACE SLAB
Ringedalsvand Dam, Norway

The cores also showed that the tightening layer was generally in good condition although it was porous in varying degrees and had definite construction joints. Since most of the leakage water, after passing through the tightening concrete, follows the drainage pipes to the inspection gallery or to the base of the dam, the lean concrete back of the tightening layer is not affected greatly by the water, except that part around the drain pipes.

REMEDIAL MEASURES UNDERTAKEN

An attempt was made in 1927 to grout the tightening layer of a short section of the dam. The grouting reduced the leakage at that point to a certain extent. In 1928 the facing joints over a small area were treated with a bituminous material. This decreased the leakage during the summer, but since the material cannot withstand much abrasive action, most of it was scraped away by the ice during the following winter. The leakage through the dam seems to have been nearly constant since 1927. This may be due to the fact that the most easily attacked part of the concrete (calcium anhydrate) had been previously carried away for the most part and that the water on its passage through the concrete is now wearing down the less easily dissolved and more resistant silicates and aluminates.

The condition of the dam was such that a complete rebuilding was necessary, and work was started in 1929. An illustration shows the downstream face of the dam with construction cranes on the crest. The reconstruction consisted of building a concrete flat-slab dam $6^1/_2$ ft. in front of the old dam and resting against it on struts. The new vertical deck is proportioned to take the full water load so that the only purpose served by the original dam is as a support.

The dam at Porjus, 30 miles north of the Arctic Circle, consists of an earth fill, two concrete spillway sections

over the natural channels, and a short concrete gravity section, as shown by the photographs. The west spillway is 270 ft. long and the east section is 378 ft., each consisting of concrete arches between concrete buttresses. The thickness of each buttress is 5 ft. and the free span of the arches is 19 ft. The tops of the arches were

REPAIRING THE FACING OF THE SPILLWAY APRON, PORJUS DAM
Reservoir Level Maintained with Wooden Needles Supported on Hinged Bents

covered with a flat concrete deck to obtain a smooth spillway surface. This deck extended over the downstream slope and was paved with granite blocks set in rich mortar.

The upstream side of the crest is inclined so that the ice will be broken up and pushed over the crest. Furthermore, a stone-protected dirt fill has been placed in front of the dam to insure against leakage. The upstream side of the arches has been covered with cement plaster and painted twice with an asphaltic solution. The east end of the spillway dam abuts on a 39-ft. roller-gate opening, which was intended to be a log sluice, but as Porjus is near the timber line, no logs pass the dam.

The dam is so located that ice pressures are not great between the roller gate and the intake on the east shore. It was therefore found feasible to build at this point a gravity section having a nearly vertical upstream face and a normal gravity cross section. The section has been reinforced by a counterfort at each side of all openings. It has a waterproofing layer on the upstream side, inside of which is a system of drains. There is a granite block facing on the downstream slope and on the upstream face up to $6^1/_2$ ft. below the normal water level. The concrete of the arches is a 1 : 3 : 5 mixture, with the exception of the 8-in. waterproofing layer on the upstream side, which is 1 : 2 : $2^1/_2$. The concrete in the deck over the arches, in the buttresses, and in the gravity section is a 1 : 5 : 7 mixture.

As in the case of the dam of similar type at Bullerforsen in central Sweden, the destruction of the dam was threatened by leakage which caused corrosion and disintegration of the concrete in both the multiple-arch and the gravity sections. The mix was too lean and leakage water leached out the lime of the concrete so that under the additional strain of frost action the dam became progressively more porous and leaked excessively. Repair work was started about 1920 and has been continued each succeeding year.

In 1929, an attempt was made to grout the concrete sections, using a pressure of about 70 lb. per sq. in. at the grout gun. Considerable leakage was stopped by the grouting operation. No grouting was done in the foundation formation. The drain pipes that allow the escape of leakage from under the over-fall domes have been regulated so that 2 or 3 ft. of water remains as a partial seal to the upstream toe. The temperature is such that no ice is formed under the domes of the dam. During the summer of 1930 preparations were being made to apply a waterproofing compound to the inside of the arches of the shorter, or northern, spillway section in an attempt to seal off the leakage. Electric heating elements were installed in this section to heat and dry the air to a certain degree. It was planned to have the entrance shaft doors at each end open to create a draft through the hollow section.

The crest of the spillway sections is regulated by timber needles 15 ft. long, supported by hinged steel bents. During the winter season the over-fall face of each spillway is nearly covered with ice as a result of leakage through the needles. Mortar in the joints of the facing, especially the vertical joints, is greatly injured by frost action. After it has been loosened in this way it is easy for the overflow to wash it out. A few blocks are washed out of the facing each year and some erosion is done at the toe in the blocky formation of the river bed. Repair to the facing is done by chipping out the poor mortar and repointing the joints with a rich mortar. The opportunity for repair to the spillway slope is small because no work can be done in the winter period, and after the ice is off the slope the flood flow must be passed.

FORSHUVUDFORSEN DAM AND POWER HOUSE, DAL RIVER, SWEDEN
Spillways Controlled by Roller Gates

The gravity dam has a very slender cross section. The downstream slope is kept moist by leakage in spite

of the attempt at grouting in 1929. The first moist spots are seen only slightly below the water level in the reservoir, showing that the concrete is very porous. Leakage is periodically measured in order to judge the condition of the concrete.

High-water level in the reservoir is at elevation 1,029 ft. above sea level. The water level is maintained at nearly a constant stage throughout the year. The minimum temperature at the site is about −25 deg. fahr.; the reservoir is ice-covered for several months each year.

SPILLWAY APRON AT FORSHUVUDFORSEN REBUILT

The Forshuvudforsen Dam, built from 1917 to 1921, is made up of four roller-gate sections, each 56 ft. wide; two gravity wing dams, one at each end; and a power house, which is an integral part of the dam, giving an overall length of 410 ft. For reinforced sections, such as piers, a wet concrete was used, but for more massive work the mix was earth-moist. The forms were hammered in all cases and all reinforced concrete was spaded. In this manner a concrete was obtained that had a dense surface without further treatment. The concrete, which was poured in the winter, was specially protected against freezing. In the summer all new concrete was sprinkled to prevent the formation of cracks.

After thorough testing, a mix of 1 : 4 : 6 (323 lb. of cement per cubic yard) was determined upon for the body of the dam. The concrete was faced with granite blocks wherever it would be subjected to the action of running water, to ice scour, or to floating logs. This facing was set into, and anchored by, steel ties to a rich face layer of concrete, of 1 : $2\frac{1}{2}$: 3 mix, which was 27 in.

LOGS PASSED THROUGH SPILLWAY DAMAGE APRON
Forshuvudforsen Dam, Sweden

thick measured from the outer surface of the facing blocks. In very massive sections of the dam a 1 : 5 : 7 mixture was used.

In 1930 there were some calcium carbonate formations at the joints of the piers. The leaching of the calcium was probably caused partly by direct precipitation on the pier and partly by seepage from the reservoir. The only actual failure was caused by erosion of the protective mat and of the rock in the river bed below the two smaller gates. For the greater part of the floating season timber is passed through one or both of these regulating gates, through which, at high-water periods, the velocity of the overflow may be as much as 25 ft. per sec. The area affected at the toe was repaired by means of a cofferdam

WEST SPILLWAY OF POROUS DAM IN OPERATION
Fill Section at Left

in the low-water season in the spring of 1930. In making the repairs, the same type of construction was used as that of the original work.

The apron consists of a timber-clad concrete mat anchored to the rock of the river bed. Below the two smaller gate openings, through which timber is floated during relatively low water, the timber deck is protected by strap iron. It has been found by experience that strap-clad timber decks can withstand the erosive effect of floating ice and logs, but that steel on concrete is unsuitable for this purpose. The heavy concrete mat is provided with transverse bearing timbers, having approximately 9-in. butts, set flush in the concrete and anchored by $1\frac{1}{4}$-in. bolts. The timber deck is laid on, and bolted to, these bearing timbers. The deck timbers have a butt diameter of about 8 in. Each deck timber, which is laid parallel to the water flow, has a 2 by $\frac{5}{8}$-in. flat iron strap fastened on top of it by 6-in. tree screws on 16-in. centers.

PERCOLATION OF WATER THROUGH CONCRETE

Without exception, it was found that deterioration occurred only in structures exposed to one-sided water pressure. Such pressure is exerted not only against any ordinary dam but also against dams subjected to water pressure on both sides, if there is even a small difference in the water levels. On the other hand, no deterioration has been found in subaqueous structures that are not exposed to one-sided water pressure, even if these structures have been made from a comparatively lean mixture.

A porous concrete allows water under the reservoir

head to enter the body of the structure and dissolve out certain components of the cement, which are deposited on the downstream slope of the dam, if the percolation is not too great. Lack of tightness in the concrete may be caused by crack formation due to the setting and shrinking of the concrete, by temperature differences during construction, or by seasonal temperature variations. If an unclean sand is used in the concrete mix, the concrete tends to become more and more porous with service. Careful tests at the old Rjukan Dam and at Ringedalsvand prove that leakage water annually carries away a large amount of solid matter from the concrete structure. Each of these dams was replaced with a new, or practically new, structure after a few years of service. A lean concrete mixture had been originally used in both cases and probably unclean sand was a major contributing cause to their rapid deterioration. Leakage from any dam indicates that slow but definite deterioration of the structure is under way and that investigations of the extent of the damage and of suitable remedies should be made at frequent intervals.

ROLLER GATE IN MAIN SPILLWAY
Porjus Dam, on the Lule River, Sweden

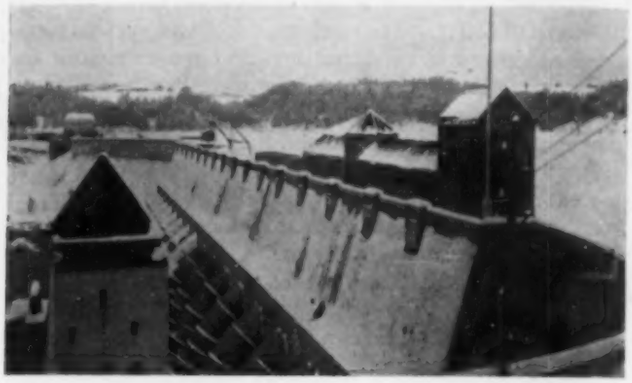
ICE AND LOG ABRASION

In addition to the compressive stresses caused by the ice cover back of a dam, ice has a considerable abrasive effect on the face of a dam, and on the spillway crest and slope if the spilled water contains ice blocks. Storage dams are affected most because from them water is released during the winter months to supplement the low natural flow. The ice cover is periodically broken up as the water level is lowered. A waterproof paint applied to the upstream face of such structures is scraped away in one or two seasons, mainly by ice abrasion. The ice cover on a reservoir in Norway or Sweden has usually become quite heavy by the time it is broken up and carried away by the early floods. As the blocks of ice are borne over the spillway, they tend to wear away the concrete on the spillway slope and toe.

In Norway and Sweden the lumber industry has been one of the chief means of livelihood for many years. The logs are cut and hauled to the streams in winter to be carried down by the spring floods to the paper mills and sawmills near the sea coast. In a normal year, from six to twelve million logs are floated down each major river. The logging associations try to obtain a separate logway, but often the log run is passed through the spillway, especially if the head is not over 40 or 50 ft. and if the spillway is controlled by a large sector or roller gate. The impact of the logs on the slope and at the toe of the spillway is so great that these structures as well as the stream bed immediately below usually must be covered with a protective mat after a few years' service, as was done at Forshuvudforsen.

TIGHT CONCRETE PREVENTS PERCOLATION

In most of the concrete dams in Norway and Sweden, especially the storage structures, the concrete has suffered more or less serious damage. Most natural water is corrosive to concrete and its destructive action is increased where freezing temperatures are experienced. Deterioration of valuable and important structures in Scandinavia has now progressed to the point where costly and extensive repairs and maintenance are necessary.



SEVERE WINTER CONDITIONS ENCOUNTERED
Vamma Dam, Glomma River, Norway

In general, the observed damage is local and does not affect the whole dam, which fact indicates non-uniformity in materials and workmanship. The damage usually starts at construction joints. Organic matter and iron and manganese combinations are generally found in the damaged concrete. The chief reason for the damage to the concrete of dams is the removal of calcium from the concrete. Cracks, caused by shrinkage and temperature variations, allow water to reach porous sections. Analyses of river water in Norway and Sweden show that in general the water is acidic and aggressive. The comparatively rapid progress of damage in many dams is due to the use of too lean a mix and to uneven mixing and placing of the concrete.

Any concrete structure that is subjected to the percolation of water of an acid nature will be slowly destroyed through the dissolving of the calcium and calcium combinations present in the concrete. The calcium anhydrate will be dissolved first and later the more resistant calcium silicates and aluminates. The speed at which the process takes place will depend upon the character of the water and the quality and cement content of the concrete. Mechanical erosion by the water will be a factor causing heavy leakage. In general, a tight, rich concrete should be used for all hydraulic construction where the water is corrosive to calcium or where freezing temperatures are experienced.

Investigating a Foundation in Soft Soil

By Means of Borings, Test Piles, and Soil Bearing Tests

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IN the expectation of spending several million dollars to erect a group of factory buildings on a site at Baltimore, Md., it was necessary to make a thorough investigation of subsurface conditions. Although the surface soil was soft, in places incapable of supporting a pile driver, or even a man, core borings revealed the presence of a thin sandstone stratum underlying about half the site at a depth of from 30 to 36 ft. Even steel-shod wooden piles generally would not pierce this stratum, which, however, was not considered sufficiently strong to carry the heavy buildings proposed. It was therefore necessary to conduct soil bearing tests and to drive test piles. In this article the results of the many tests made are analyzed, and their use, as a basis for selecting the type of foundation chosen, is explained. In addition, comparative estimates are given of the cost of foundations supported on wood and concrete piles. The paper was originally presented on January 21, 1932, before the joint session of the Construction and Structural Divisions at the Annual Meeting of the Society.

IT is almost impossible to get usable records of the settlement of pile foundations, for even when such settlement has apparently taken place, the question arises as to what was the original elevation of the structure. In general, there is a reluctance to permit publication of anything implying that a foundation is unstable. In one case there is evidence that the pile foundations under a large building have settled as much as 3 in. in 15 years, yet even this may not be cited for fear of injuring the reputation of the building. In other cases buildings have developed cracks that can be attributed to unequal settlement of the pile foundations.

Pile foundations, like spread foundations, are subject to settlement, and this fact should be fully realized when they are being designed and constructed. The difficulties attending the supervision of pile driving are such that it is nearly impossible to be certain that all the piles are driven to a predetermined resistance. Even under constant supervision there are likely to be wide differences in the depth of penetration and final resistance of the individual piles in any one group, and even greater differences between different groups on the same job.

FOUNDATION INVESTIGATION STARTED

During the fall and winter of 1929–1930 an investigation was made of foundation conditions at a site on which it was proposed to erect a large group of six-story buildings for the Point Breeze Works of the Western Electric Company, Inc., near Baltimore, Md. The building site, shown in Figs. 1 and 2, is near the mouth of the Patapsco River, and is 825 by 875 ft., or about 17.5 acres in extent. The initial construction is expected to cover the eastern third of this area, on which section most of the investigation was concentrated.

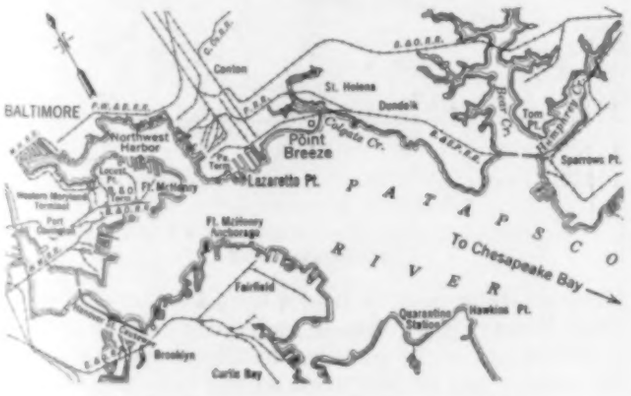


FIG. 1. LOCATION OF POINT BREEZE, MD.

Test borings revealed an unusual subsurface condition, the presence of a thin stratum of so-called sandstone about 25 ft. below the Baltimore city datum, and extending under about half the area of the site. The thickness of this stratum was recorded as varying from 3 to 30 in., and averaging less than 14 in. With moderately hard pressure, the point of a pocket knife will penetrate a sound core of this "sandstone" $^1/_{16}$ in. or more.

To determine the bearing power of the ground, several soil tests were made by loading areas 2 by 2 ft., at an elevation 4 ft. above datum, with test loads equal to 8,000 lb. per sq. ft. Settlements ranging from $^1/_2$ to $3^1/_2$ in. were recorded. Probably these bearing tests would not fairly represent the actual bearing power of the ground if the high groundwater level were lowered by sewers and deep excavations. Spread footings seemed impractical for this project.

A series of 56 wash borings and 7 shot-drill core borings were made, and 8 test piles were driven. The core borings extended approximately 100 ft. below the ground surface, or from 88 to 96 ft. below zero datum, which is mean low water. The other borings were carried to depths varying from 22 to 59 ft. below datum. The records of the borings and test piles, as well as the samples of borings, were submitted for study and report prior to the driving of 41 additional test piles.

A study of the records of the borings and test piles disclosed the fact that two of the three original test piles driven within the area of the proposed initial construction went down to the "sandstone" only, and one apparently was driven through the "sandstone" and 17 ft. below it.

A few characteristic boring records have been plotted to scale and are given in Fig. 3 with the designations of the different materials as recorded in the field, and

numerals indicating "the number of blows of the hammer weight falling 12 in.," required to make the sample spoon penetrate 1 in. The hammer weight used for each boring, as indicated by "H. W.," ranged from 120 to 160 lb. In studying the records of the borings, particular attention was given to those located within the area of the proposed initial construction and adjacent to it.

TEST LOAD ON A CONCRETE PILE
60 Tons of Pig Iron and Lead Caused Settlement of $^3/_{16}$ In.

The stratum of "sandstone" was struck in 23 of the 27 borings, at from 20 to 28 ft. below datum. From the records of all the borings it appeared that the "sandstone" stratum lay under about one-half the site and under nearly the entire eastern third, as indicated on the plot plan in Fig. 2.

As shown in Fig. 3, the recorded character of the material above and below the sandstone stratum was such that evidently wood piling could be driven down to the top of the stratum with comparative ease, and if the "number of blows of the hammer weight" on the boring spoon was any criterion, then in most cases piles could be driven readily through the material below the "sandstone" stratum, provided they could or would penetrate this stratum.

GEOLOGY OF SITE STUDIED

In Baltimore County, most of the higher elevations along the inner margin of the Coastal Plain are capped with the deposits of the Patapsco formation, which consists generally of highly colored, variegated clays, interbedded with sandy clays, sands, and gravels. In many places the sandy beds in the vicinity of the clays are indurated to form layers of ironstone. The iron-ore clays are confined to the upper parts of this Patapsco formation. The iron from these beds, in the form of an oxide, filters down through the underlying sands to a great distance, staining them a deep red color. When the infiltrate reaches a clay layer, the iron oxide is strained out by the clay, which it thoroughly permeates, often imparting brilliant hues. These deeply stained clay layers are universally known as "paint stone."

This paint stone, or red ocher, is not uncommon in the district immediately south of Baltimore City. It often consists of a very sandy clay, upon which the effect of the iron has been to form a more or less indurated crust, sometimes a true rock. Apparently "paint stone" is the material shown in the records of these core borings as "sandstone," and this conclusion is confirmed by chemical analyses.

In an effort to discover the origin and character of the layer of so-called sandstone underlying most of the Point Breeze site, an inspection was made of similar formations in the neighborhood, among them an exposed stratum of "red sandstone," reported to be characteristic of this deposit in the vicinity of Baltimore. It had been exposed in making an excavation for Hanover Street near the end of the Hanover Street Bridge and several miles from the Point Breeze location. The notes made at the time of the inspection read:

"It is an undulating stratum in wave-like formation, varying in thickness from 1 in. to 24 in. and of such hardness that it would be impossible to penetrate it with a pile point. The materials below this stratum are similar to those disclosed in the borings at Point Breeze. Samples taken of this hard rock-like material appear quite unlike any of the boring samples (from Point Breeze), but the latter may have been disintegrated by the boring or drilling tool."

The specimen of rock obtained from the cut on Hanover Street made just previous to the investigation at Point Breeze, is evidently ironstone, a siderite in a more or less impure form admixed with clay and limonite.

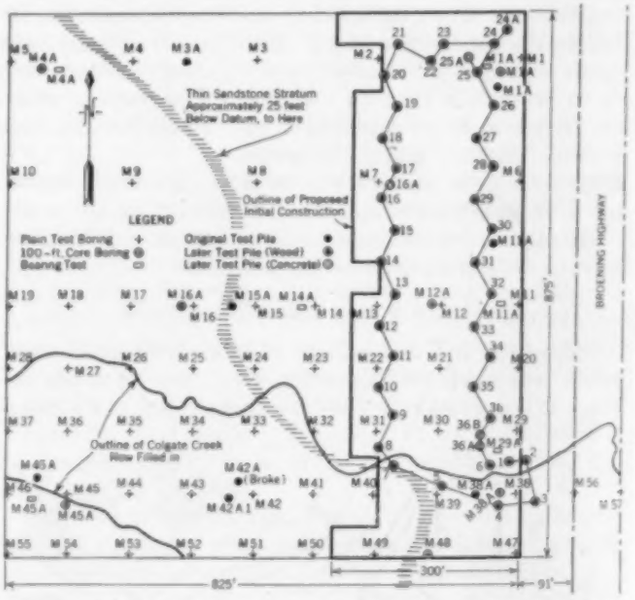


FIG. 2. PARTIAL PLOT PLAN OF POINT BREEZE WORKS OF THE WESTERN ELECTRIC COMPANY
With Location of Test Piles, Borings, Soil Bearing Borings, and Bearing Tests

However, there is no evidence from the core borings at Point Breeze that ironstone underlies that site.

The weight of the proposed buildings, together with a fair assumption for live loads on the various floors, was estimated to total approximately 1,000,000 lb. per

column footing, if the columns are spaced 25 ft. apart. The assumed live load on the ground floor was 500 lb. per sq. ft., not carried to the columns. Consequently, the imposed load would average approximately 2,100 lb. per sq. ft. over the entire site. In addition to this, the weight of 3,700 lb. per sq. ft. of earth above the sandstone stratum would make the average load on this stratum 5,800 lb. per sq. ft. Since the building loads would be concentrated on footings, probably supported by groups of piling, the loads per square foot on the sandstone stratum would vary from 4,200 lb. on areas between pile groups, to 7,700 lb. on areas directly under pile groups, provided the piles extended down to, but did not penetrate through, the "sandstone" stratum.

ADDITIONAL TEST PILES NECESSARY

As an investment of several million dollars was proposed on this site, it seemed desirable to determine definitely whether or not piles could be driven through the "sandstone" stratum. Consideration was given to experimenting with (1) steel-shod wood piles, (2) steel-pipe piles, and (3) thin steel pile shells, driven with a removable core.

It was decided to drive a series of steel-shod wood piles at selected column centers. In Fig. 2, the length of the zigzag line connecting the selected pile centers was approximately 1,800 ft. There were 41 test piles driven in 11 working days. A large part of the time was required to move the heavy pile driver over the soft and muddy ground. In all, 37 wood piles were driven, one being discarded as being too short to drive to refusal.

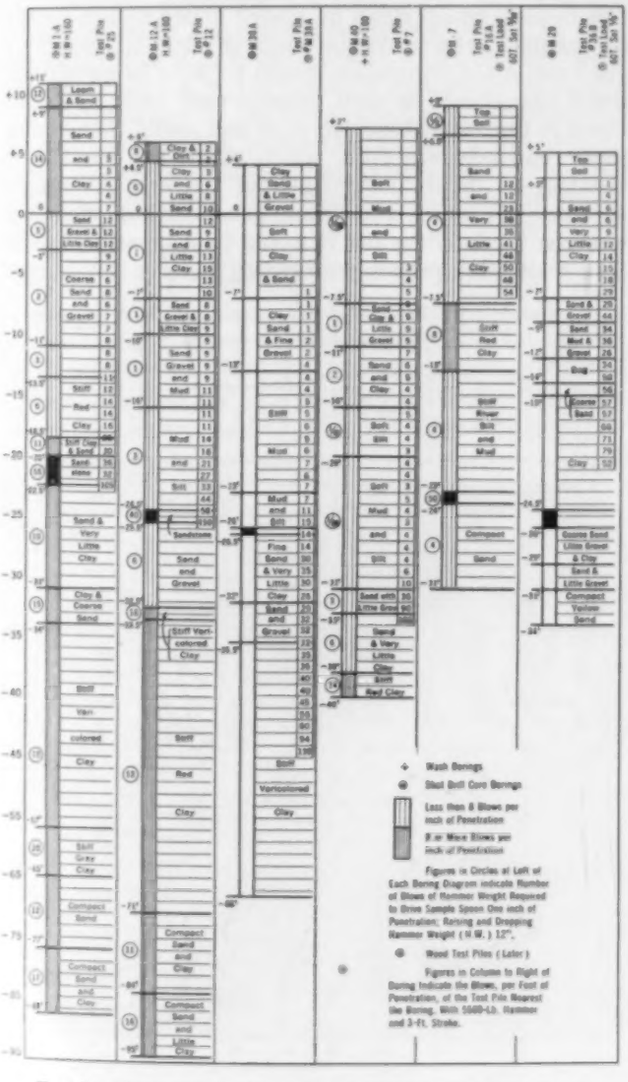


FIG. 3. SELECTED RECORDS OF BORINGS AND TEST PILES

MAKING A TEST BORING ON THE BUILDING SITE
Boring M-41

Four concrete pile shells were driven, one of which was discarded when a section of the shell collapsed after the driving core was withdrawn. The wood piles were all carefully selected and no pile was used that would not meet a rigid specification for straightness and size. Generally the wood piles had 8-in. points and 14-in. butts.

The four concrete pile shells were all standard except one, which was designated as a "stub pile." The pile shell that collapsed was not concreted, but the other three shells were filled with a 1:2:4 concrete having a 3-in. slump and made with quick-setting cement. The total cost of the 37 wood piles and the driving of the 41 test piles (including driving the 4 steel shells) was $7,044, or $172 per pile driven.

PILE DRIVING RECORDS ANALYZED

In Fig. 3, a direct comparison between the recorded number of hammer-weight blows on the boring spoons, per inch of penetration, and the number of pile-driver blows on the test piles, per foot of penetration, shows that the former should not be considered as indicating the resistance of the ground to pile penetration, regardless of what other value it may have. In some places where the boring spoon went down very easily, resistance to the driving of piles was very great.

An analysis of the pile driving records of the 39 wood test piles has been made, and the results are given in Fig. 4. On this diagram has been plotted the average

number of blows per foot required to drive each pile through every foot, from 1 ft. above datum to 24 ft. below datum. A curve has been drawn which represents the mean of the average points. This average curve has been compared with the *Engineering News* pile-driving formula, and the "safe loads" according to that formula are shown on the graph. It is evident that the average pile would have to be driven down to about −17 in

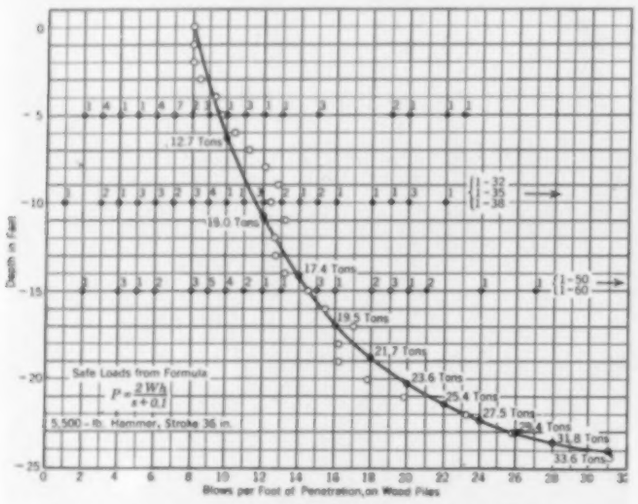


FIG. 4. DRIVING RECORDS OF 39 WOODEN PILES
5,500-Lb. Hammer, 36-In. Stroke

order to safely carry 20 tons according to this formula. However, there is no such thing as an average pile, even in this limited area, because the number of blows per foot at each foot of depth varied between very wide limits. This is shown by the points plotted on the −5 ft., −10 ft., and −15 ft. lines.

It is evident that the records of one or two test piles driven at any site may give very misleading information. There are even more variables in an ordinary set of pile driving records than in the records of this unusually large number of test piles, where each pile was carefully selected for size and straightness. It shou'd be borne in mind that nearly all these piles had steel shoes or points, although there is no noticeable difference in the driving records of piles with and without shoes. The purpose of using steel shoes was to increase the chances of penetrating the "sandstone" stratum.

RESULTS OF LOAD TESTS ON PILES

Load tests were placed on two of the wood piles and three of the concrete piles. The total test load on each wood pile was 45 tons, and that on each concrete pile, 60 tons. These loads remained on the pile until no additional measurable settlement took place in 24 hours. The records of these five load tests are shown in Fig. 5.

There were evidently instrumental errors in the readings for the first load test, that on wood pile No. 16. The other readings were correct. Under the 45-ton load, a settlement of approximately $^3/_{16}$ in. was noted for wood pile No. 36, which had been driven to refusal at minus 26 ft. The other wood pile apparently settled about the same amount, although it was purposely not driven to refusal but only to −21 ft. The three load tests of 60 tons on the concrete piles resulted in a settlement of approximately $^5/_{32}$ in. in each case, although the depths of penetration below datum were, respectively, 7 ft., 13 ft., and 21 ft. In all cases the surface of the ground was at least 8 ft. above datum. The results of all five load tests were considered entirely satisfactory.

DECISIONS REACHED

After all the test piles had been driven and the records studied, certain conclusions were reached. Piles driven from the surface of the ground at a grade of approximately + 9 ft., without being cut off as required for permanent foundations, had the benefit of the surface friction above datum to assist in carrying the test loads. This assistance would not be available for wood piles, which would have to be cut off at datum. Concrete piles, on the other hand, could be "cut off" at the bases of footings, in many cases above datum, and in some cases as high as +7 ft. In many instances, it would be necessary to cut piles off at depths ranging from 2 ft. to 14 ft. below datum for basement or under-grade passageways and elevator pits, leaving very short lengths of piles above the "sandstone" stratum.

It was therefore decided that: (1) if the cost of concrete piles extending above datum would be less than that of wood piles cut off at or below datum, then foundations which could be placed above datum should be designed to be carried on concrete piles; (2) wherever the base of the foundation would be at or below datum, wood piles should be used, because they could be driven down to the "sandstone" stratum and would therefore be safer than concrete piles, which might not reach to a satisfactory depth if driven from the original ground surface; (3) where the "sandstone" stratum does not underlie the building site it would be necessary to drive long wood piles, probably exceeding 50 ft. in length; and

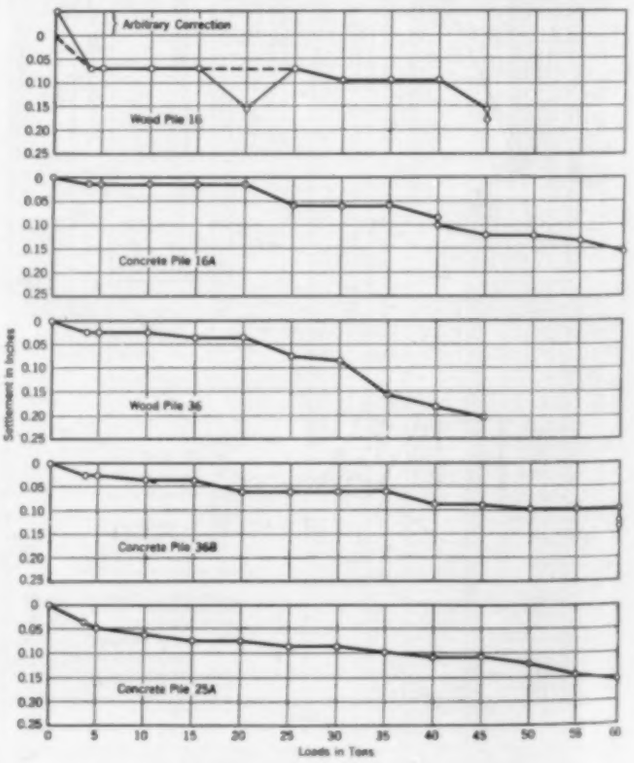


FIG. 5. LOAD SETTLEMENT RECORD ON PILES

(4) estimates should be made based on preliminary studies for the proposed buildings and on very approximate column loads, but taking into account the proposed basement area and the underground passageways and elevator pits. In Fig. 6 is shown part of a cross section through a proposed building.

ESTIMATED COST OF FOUNDATIONS

The estimates made on this basis showed that the cost of 360 foundations designed for wood and concrete piles would be as follows:

On wood piles only, at 20 tons each	$533,800
On concrete piles only, at 25 tons each	526,450
On concrete piles only, at 30 tons each	441,900
On a combination of wood piles at 20 tons, and concrete piles at 25 tons each	539,250
On a combination of wood piles at 20 tons, and on concrete piles at 30 tons each	482,800

The fourth and fifth items of this estimate are for wood piles under all footings where the pile cut-off must be at or below datum, and concrete piles for all footings where the cut-off can be above datum.

The advisability of assuming 30 tons per pile as a safe load on concrete piles for this location was doubted, notwithstanding the very satisfactory showings of the load tests on three concrete piles. The fact was appreciated that in many cases the cut-offs would be considerably below the ground surface, where the load tests were made, and that the concrete piles could be driven to a much less depth than the wood piles. Consequently the excavation of the supporting ground above +8 ft. would reduce the skin friction on the shorter concrete piles proportionately more than it would on the longer wood piles.

BASIS OF FOOTING DESIGN

In order best to balance the loads between interior and exterior footings, it was decided to design the footings on the following basis:

The entire dead load plus one-third of the live load carried by the lowest column, plus the dead load and one-third of the assumed live load on the ground floor (not carried into the columns) directly over the footing. The allowable loads on piles will be 18 tons for wood piles and 24 tons for concrete piles; and the spacing of the wood piles will be 2 ft. 9 in. on centers, and that of the concrete piles, 3 ft. 4 in. on centers. As for those footings that may be over the edge of the "sandstone" stratum, it is hoped that all the piles under them can be driven to refusal, for otherwise the problem will be complicated. Wood piles are proposed for the area not underlaid with the "sandstone" stratum.

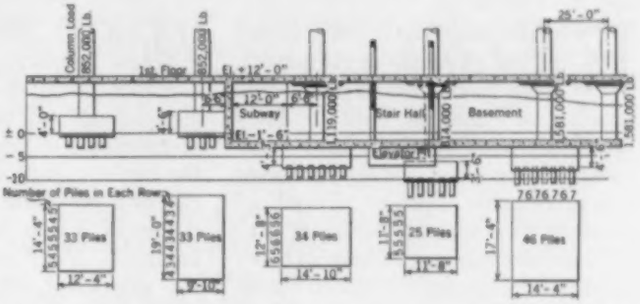


FIG. 6. PART OF CROSS SECTION THROUGH PROPOSED BUILDING Showing Types of Pile-Supported Footings Required

ACKNOWLEDGMENTS

Acknowledgment is made of the valuable cooperation of Henry G. Perring, M. Am. Soc. C.E., in connection with this investigation. His wide experience with foundations in the neighborhood of Baltimore led him to believe that it would be impossible to penetrate the "sandstone" stratum with wood piling even though fitted with steel points, and the results proved his belief to be correct. Acknowledgment is also made to O. C. Spurling, M. Am. Soc. C.E., Engineer of Plant of the Western Electric Company, Inc., for his permission to present here the information gained for his company by this investigation.

Courtesy Curtiss-Wright Flying Service

FACTORY SITE FROM THE AIR
Initial Building Site and Plot Plan Outlined

Leadership in Economic Progress

An Opportunity and a Challenge to All Engineers

By Alfred D. Flinn

Member American Society of Civil Engineers

Director, The Engineering Foundation

IN DEVELOPING means for the efficient production of wealth, has the engineer failed to understand his real significance? Should he not now take a responsible part, together with the banker, the business man, and the economist in effecting economic adjustments so that the products of industry may be properly utilized? Dr. Flinn believes that this is the engineer's responsibility and that engineers have the ability and the vision, in cooperation with men of other vocations, to discover and point out new industrial and economic procedures consistent with man's expanding creative ability and rising standard of living.

ENGINEERS have led industry in its physical development. They have created its modern production management. They have improved methods and processes. They have greatly increased the productive capacity of civilized communities and at the same time reduced the amount of human labor required. They have reduced waste. They have increased wealth and leisure, improved health, and provided means for better living. Tardily they are realizing that for the good of mankind and the benefit of their profession they should have been forethoughtful also of the changes in ways of living that their progress in the physical and managerial aspects of industry was causing. Some engineers should have devoted their energies to preparing society for the political, economic, and sociological advances being necessitated by industrial progress. Engineers possessed of the requisite capabilities should have found a demand and compensation for this form of professional service.

Economic and sociological difficulties caused in part by the failure of engineers to do their full share in the past, press more painfully than ever for correction. Obligation and opportunity are before the engineers who have, or who can acquire, the knowledge, the understanding, the vision, the sympathy in collaboration, which are necessary. The situation calls for humility as well as ability. The engineer cannot perform the needed service alone. He must not be boastful of his own contribution no matter how important or successful. He must be appreciative, also, of the contributions of men of other vocations. Whether he retains an engineering title or accepts one of the executive variety, will be unimportant. It will, however, be essential for engineers who undertake the proposed service to retain their conviction of the inexorability of the laws of nature, both physical and spiritual, with all that this conviction, gained out of experience, implies in business practices as well as in engineering.

ENGINEERING APPLICATION OF ECONOMICS

What does the word "economics" mean? Its derivation is interesting. It has two Greek parents, "oikos," house, and "nemo," manage, whence comes "economy." A few meanings of economy in the Standard Dictionary are: "disposition to save; freedom from extravagance or waste; practical adjustment, organziation, or administration of affairs or resources, especially of industrial resources of a state." This last definition lets the engineer into the house, does it not, along with other constructively minded members of the community? So much for the noun. Its adjective has for one of its definitions: "pertaining to money matters or wealth, or to the means and methods of living well and wisely." We should see engineering in both branches of this definition. It is there.

An engineer is a person trained to organize, direct, and utilize men and other resources of nature by the application of scientific principles and methods and of the mechanic arts, in order to get results desired by individuals or groups of men. An engineer must have sound knowledge of the facts and theory of mathematical and natural sciences, of human abilities, of materials and machines, along with an understanding of the character and use of wealth, both of nature and of society.

The term "engineering" has in the minds of many persons a content vastly greater than the definition of a vocation, no matter how broadly stated, greater than the phrase sometimes used as a synonym, "application of science." In fact, "engineering" now often signifies a new system of thought, a fresh method of attack upon the world's problems, the antithesis of traditionalism, with its precedents and dogmas.

ECONOMIC FUNCTIONS AND RESPONSIBILITIES

Engineering is a system of thinking that comprises analysis and clear statement of the problem; assembly and study of all available knowledge; supplementation of this knowledge, if need be, by research; deduction of tentative conclusions; correction or confirmation of these conclusions by various tests; application of the accepted conclusions to useful ends, the expenditure of energy upon each step in the solution of the problem being controlled by an appreciation of the relative economic or human importance of the results desired. Observe that "useful application" and "economic perspective," together, are the elements of this definition that differentiate the activities described from those of other vocations, and make them engineering.

Many persons cling to the fallacy that bankers and major business executives have more to do about wealth than any other men. Let us grant that they have a great deal to do about it and that because of their capacity for acquisition and accumulation many bankers and business men do gather great stores of material goods and liquid tokens of wealth. Let us acknowledge also that because of these very capacities many of these men per-

form great services for mankind in connection with conservation and construction. Whether or not some of them, taking advantage of their position near the treasury, collect too large fees for stewardship, is largely a matter of evaluation of services and of rewards. Before judging these men harshly, let us look broadly and deeply at compensations of all kinds, the satisfactions of life over its whole range—sensuous, moral, spiritual, chronological—and let us take into the reckoning the risks, the anxieties, the losses, the responsibilities of these men, with whom engineers must collaborate.

Although bankers and business men control credit, do most of the handling of money and other tokens of wealth, and perform numerous other important functions having to do with wealth, engineers perform the basic economic functions in many enterprises for the creation of wealth or for its use. The engineer's determinations of location, of size or capacity, of process or method, of materials for construction, of raw materials for manufacture, of equipment, of personnel; his design, specification, inspection, supervision—from inception of a project to marketing of products or services—underlie all the features of the business to which, collectively, the term "economics" commonly is applied. In consequence, great are the economic influence and responsibility of the engineer.

Is not economics closely similar to mathematics in that it is a body of knowledge of which, in one form or another, various parts are necessary for numerous vocations? Architects, engineers, financiers, industrialists, merchants, and others necessarily use economics as part of their equipment, as they also use mathematics and other branches of knowledge. Therefore selected men in the vocations named are competent to make certain kinds of studies involving economics.

Such studies are being made, and more are needed. Lessons learned should be effectively put to use. Engineers must do their part; and it is a much larger part than many engineers and others have conceived. They should, nevertheless, expect those in other vocations to do their parts also. Engineers who were "born that way" should fit themselves for this new service. Charlatans can only do damage.

TODAY'S PROBLEMS

Is the civilized world today struggling with two major problems, or groups of problems, and confounding them?—(1) business cycles, now complicated with many unprecedented social, industrial, financial, and political conditions; and (2) a social and economic evolution, vaster and more profound than the so-called industrial revolution. This evolution is being caused by the accumulated and continuously cumulating and accelerating discoveries of science and their applications through industry, agriculture, chemistry, engineering, medicine, and other arts, and by the continuing processes of the industrial revolution.

Engineers have been largely instrumental in bringing to the world with great rapidity the economic potentialities of the "power age," as Michael Pupin has so aptly named our era. They have not at the same time warned the world clearly and loudly enough that new business and social practices, and new economic policies were rapidly being made necessary. Apparently engineers thought that if they did a good job in creating means for the adequate production of wealth, the rest would be taken care of by the economist and the business man. The engineer and his colleagues in the application of science were the only ones who could apprehend the significance of the new progress, but they either did not take the time to "follow through" its effects, or else they shirked their responsibility, were too modest to take the lead, or were too badly overworked in developing the productive capacity of the machine age to give sufficient attention to the increasing of consumptive capacity and to human adjustments. Consequently our modern world is out of balance, almost upside down in its economics, and it does not yet perceive just what has happened.

A GREAT OPPORTUNITY

For each individual, his own economic status is of crucial concern; nevertheless he must bear in mind that the status of his group has a governing, although often subtle, influence upon all personal situations. Consequently enlightened self-interest should suggest to each individual that he exert himself to exalt the status of the group. There are times, however, when all self-interest of the individual and of the group should be subordinated to a great human need, with its opportunity for service.

These passing months are offering to engineers, to American engineers in particular, an opportunity such as never before was presented to any group of men. The world greatly needs leaders of a new type, unfettered by precedent, straightforward in deduction from proved data to objectives, honest in determinations, fearless in decisions, courageous and prompt in action, and resourceful in difficult situations. With all these qualities of manhood, these leaders must possess a clear comprehension of this new era into which engineers have so swiftly brought mankind—with the collaboration of scientists on one hand and of industrialists on the other.

Who other than the keen-visioned, clear-thinking, broad-minded men among the scientists and the appliers of science through engineering and industry can help their fellow men to understand the scientific, the engineering, the "power" age? Who better than they can supply the new element in leadership for which confused humanity is groping?

To engineers is opened now a great opportunity to bring into cooperation with themselves the scientists, the industrialists, the constructive men of all vocations, in order to develop and to promulgate a new philosophy of life and new procedures based upon this philosophy, in keeping with man's new powers, and his nobler conceptions of himself, his environment, and his Creator.

To men still using the mental processes of the hand-tool era, this mechanized civilization and its procedures appear strange and complex. To engineers and chemists who create modern industrial plants with hundreds of "almost human" automatic machines and intricate chemical processes, comprehension of a mechanized order of society is relatively easy. These men have multiplied man's capacity to satisfy his needs and desires beyond the dream of Aladdin; they have perfected means for transportation, for communication, for recording. Now they must "follow through," or at least lead on.

We have too much legislation; most of it should be scrapped. We need to know and to understand laws.

Laws exist in nature and are immutable. Statutes may be flouted, but laws cannot be evaded. It is the task of research-minded scientists, engineers, business men, and lawyers to discover more of nature's laws, to state them correctly in language that the peoples can understand, and then to spread abroad knowledge of the laws as they are learned, so that the peoples may live happily in obedience to laws, not harried by the "thou-shalt-nots" of legislation.

A NEW PHILOSOPHY FOR INDUSTRY AND TRADE

To have a more nearly fair division of the wealth which can be so abundantly produced, and to do away with the wild changes in business which bring periods of extravagance and of distress one after another, the practices of industry and trade must be guided by basic principles embedded in the nature of man. Some of these principles have been known for thousands of years and have been stated to all the races of men by their great teachers. Because of over-reaching selfishness, men have largely disregarded these principles in business and in political relations. Humanity continues to suffer the consequences of this stupidity.

Men cannot continuously live happily and safely in civilized communities if they disregard the principles of fair play, honesty, and truthfulness in act and word any more than they could have machines, power, railroads, electric lights, telephones, automobiles, radios, better health, and innumerable other benefits from modern science, if chemists, doctors, and engineers had not been guided by the principles of gravity, heat, electricity, chemistry, and biology.

Knowledge of the basic principles of human nature and of physical nature is incomplete; there is much to be learned. Diligent seeking for truth will add to knowledge. Practical use of knowledge in all human activities will add to well-being and the enjoyment of life, on a steadily rising plane. A relatively small number of individuals may prosper for a time in defiance of the principles of human nature, but they do so at the cost of society as a whole. The masses—all of us—suffer also the consequences of our ignorance and greed. There is no escaping the penalties for going contrary to principles; they will come upon us sooner or later.

OPPORTUNITY AND OBLIGATION

This is then the engineer's opportunity and his great obligation—to lead constructive, forward-looking men in establishing the new economic status for all peoples of the earth on the basis of scientifically discovered laws.

Along this route lies the way out of the wrong basic conditions that make business depressions possible. To establish the new economic status will take much time. For one of the reasons recall those words of the Gallilean: "Oh fools and slow of heart to learn all that the teachers have spoken." Let us hope that meanwhile, however, humanity will find various expedients for ameliorating its misery, at least for a short time. To expect that there will be no more business depressions is not yet sensible. So long as men are so stupid as to try to get rich too quickly—generally at the expense of the helpless or the guileless—there will be other depressions, for as Kipling inimitably wrote ten years ago in his poem, "The Gods of the Copybook Maxims":

"And the burnt fool's bandaged finger goes wabbling back to the fire."

There is a new job for members of the "fifth estate," defined by Arthur D. Little, a leader among chemists and a former vice-chairman of the Engineering Foundation, as being "composed of those having the simplicity to wonder, the ability to question, the power to generalize, the capacity to apply. The company of thinkers, workers, expounders, and practitioners upon which the world is absolutely dependent for the preservation and advancement of that organized knowledge which we call science." Upon the "fifth estate" depend also the maintenance and upward progress of civilization.

A COVERED BRIDGE IN EUROPE

Century-Old Structure in the Italian Alps, Ticino Valley

Design of Tunneled Sewers in Unstable Ground

Principles of Soil Pressure and Practical Methods of Design

By Robert G. Hennes

Junior American Society of Civil Engineers

Formerly with the Bureau of Tests, Department of Public Works, Detroit, Mich.

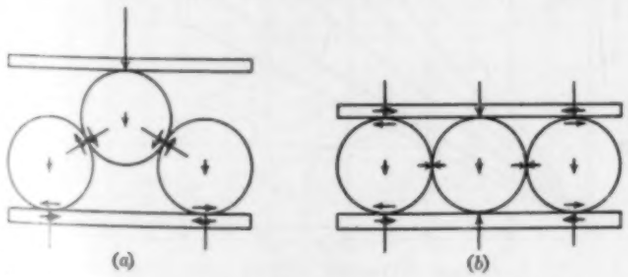
IN the past there has been a great variation in the design of circular arch ribs for sewers excavated in tunnel, because no rational basis had been developed for the design of such structures. In this article Mr. Hennes explains the theory of arch action in soils as applied to this type of construction, offers a simple test for determining the ratio of the lateral to the vertical soil pressure, and presents diagrams for designing the lining of circular sewer tunnels to withstand expected soil pressures.

RAPID growth of urban population and rising standards of living created so urgent a demand for sewerage systems that engineers were given no time for the development of a scientific basis of sewer design. Nevertheless, the early sewers were quite satisfactory. Repairs, when needed, were made without much inconvenience to the public or conspicuous surface activity, and in consequence there was no public clamor for better sewers. It was natural that progress in the design of tunneled sewers, from the structural standpoint, should lag behind that made in other branches of construction. Of two sewers built under similar conditions, one might be of reinforced concrete adapted to resist considerable bending moment, while the other would be a brick ring designed for purely compressive stress. It is not wholly unreasonable therefore to approach the problem of sewer design as though it were entirely new, and to seek a solution through the application of our present knowledge of soils and technical mechanics.

Where the surface is level and the soil undisturbed, the vertical pressure at a given depth is equal to the weight of the superincumbent soil. The horizontal pressure bears a constant ratio to the vertical pressure, at least for ordinary depths. The first assumption is the only reasonable one possible; the second has been demonstrated experimentally.

SOIL PRESSURES ARCH OVER TUNNEL EXCAVATION

If a tunnel is driven through the subsoil, the weight of the superincumbent soil is transferred to each side by arch action. The strength of the soil arch depends upon the nature of the material. In a soil having very little cohesion—clean coarse sand, for example—such an arch would be unstable if the intrados were left entirely unsupported, because even the impact of traffic or the flow of groundwater would break up the arrangement of the particles. An idealized picture of arch action developed in three spheres, or cylinders, is given in Fig. 1*a*. The middle particle maintains its position because of the frictional force developed by contact with the neighboring particles, and transmits its load to them. If the equilibrium were destroyed by impact, or if the coefficient of friction were lowered sufficiently at the points of contact—as would happen if a bit of oil were dropped at these points—then the arch would fail, and the particles would rearrange themselves as in Fig. 1*b*, each carrying a third of the load, and exerting a horizontal pressure determined by Poisson's ratio.



(*a*) (*b*)

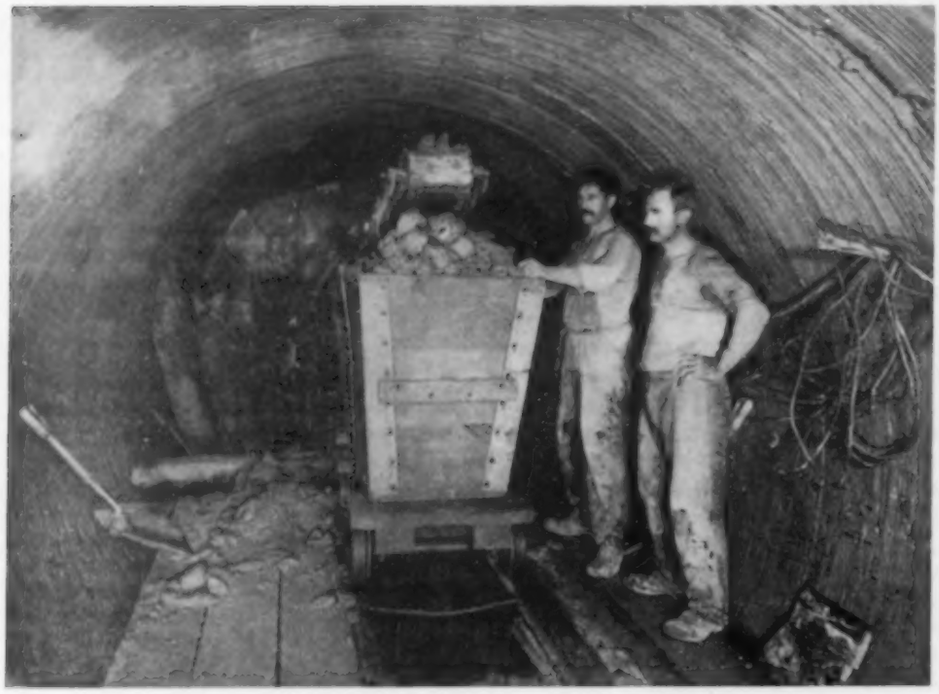
Fig. 1. Theoretical Arch Action in Perfect Spheres

Reynold demonstrated by experiment that the rearrangement of the grains in a compact mass of sand involves a temporary increase in the volume of the voids.

Mining Operations with Draw Knives and Jack Spades
Diameter, 14 Ft.; Lining, 16 In. Thick; Daily Progress, 10 Ft.

This means that if the expansion of the soil around a sewer is prevented, the sand arch is stable. Thorough grouting would probably prevent this expansion and so make the soil arch dependable. If, however, cavities are left around the tunnel lining or if they develop as a result of leaks in the barrel of the sewer, then the failure of the sand arch becomes a possibility.

As regards arch action, clay differs materially from sand. The compressive strength of laterally confined sand is equal to any load to which it is likely to be subjected, but clay is often stressed beyond its strength. In deep workings the phenomenon of "swelling clay" is frequently observed. This is generally a manifestation of earth pressure alone; it is caused by the flow of the overloaded clay into the bore. In such a case the arch action is negligible or non-existent. On the other hand, when the swelling is not an actual flow of material, but



TUNNELING MACHINE OPERATING IN SELF-SUPPORTING SOIL
Lawton Avenue—Morrel Street Sewer, Detroit, Mich.

represents the elastic expansion of the clay on its release from a pressure to which it has been long subjected, then the arch must be in operation, at least temporarily.

PRESSURES IN SATURATED CLAY

The researches of Terzaghi have shown that the compression of a saturated clay is effected by expelling part of its water content. This means that the water content of a given clay, after it has attained equilibrium, is a function of the weight of the overlying soil. If then

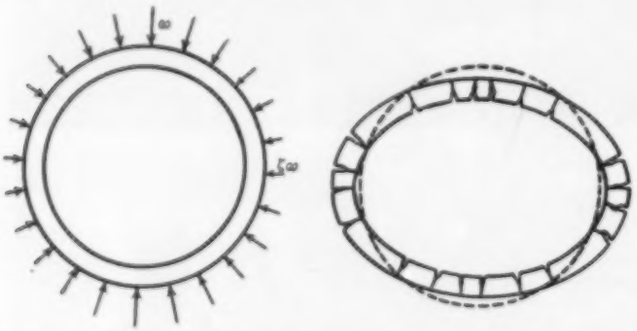


FIG. 2. RESULT OF UNEQUAL RADIAL PRESSURES ON A PERFECT ARCH RING

a tunnel be driven through a saturated clay that is in equilibrium, the additional pressure that is carried to the adjacent body of clay by the thrust of the soil arch will result in an excess of hydrostatic pressure in the compressed parts. This pressure, transmitted hydraulically to the soil in contact with the lining, causes a redistribution of stress, and at least the partial cessation of arch action. This reasoning applies only to saturated clay, or to clay so nearly saturated that the additional compression caused by arch formation results in the development of hydrostatic pressure. It is, however, conceivable that even in stiffer clays vibration and impact such as result from highway traffic might cause a temporary pressure greater than the compressive strength of the soil, with consequent breakdown of the arch.

Undoubtedly, the problem of tunnel design is complicated by such changes in pressure conditions underground. It is apparent that an unlined tunnel cannot be driven without changing the pressure conditions in the soil. It is less apparent that after the tunnel has been lined there is a further readjustment of stress in the surrounding soil. Nevertheless considerable evidence points to such a conclusion, even for solid rock. For instance, mining towns often suffer from ground settlements due to the closing of abandoned drifts. Many instances are known of tunnel walls slowly coming together and even of inverts rising.

Another difficulty that can be satisfactorily explained by the gradual mobilization of earth pressure is described in an article in *Engineering News*, Vol. 71, page 84, by Otto Hufeland, on the valuation of old brick sewers in Manhattan. Examination showed that the life of the average brick sewer is about 64 years and that its deterioration may be divided into seven stages, which may be characterized as follows: (1) cement partly out at the water line; (2) cement partly out above the water line; (3) depressed arch and sewer slightly spread; (4) large open joints; (5) loose brick; (6) bond of brickwork broken; and (7) distorted sides, uneven bottom, joints out of line. This last condition is very

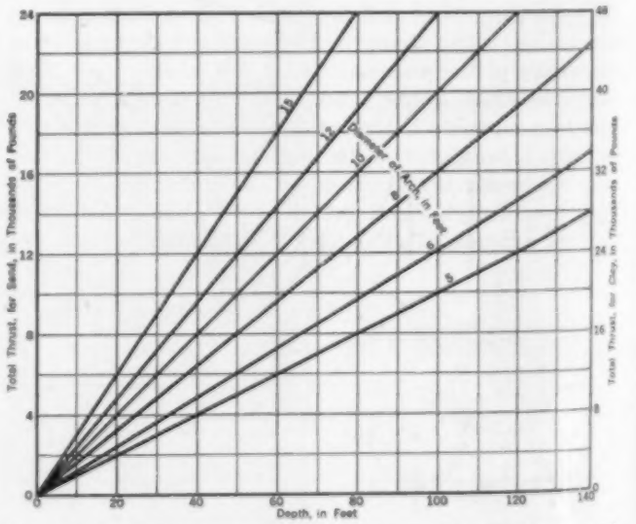
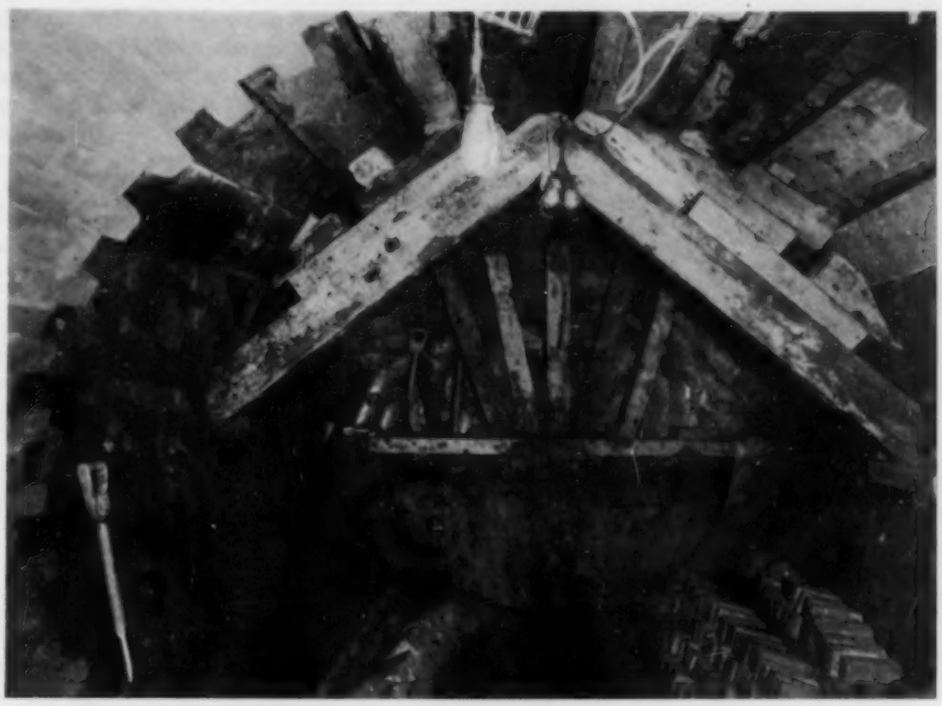


FIG. 3. RELATION BETWEEN DEPTH AND TOTAL THRUST IN TUNNEL ARCH RINGS, FOR SAND AND CLAY

similar to the deformation that would result from a gradually developing system of unequal radial pressures, as pictured in Fig. 2. Of course the durability of the structure depends upon its strength as well as upon the pressures to which it is ultimately subjected. Moreover, such factors as alkali groundwater or acid sewage may ruin masonry before external pressures are fully mobilized; and abnormal stresses due to nearby tunnels or building foundations may have a greater effect than the normal earth pressure.



BRACING SYSTEM ONCE IN COMMON USE IN FAVORABLE SOILS
"Horsecups" and "Mudsills" Followed by a Brick Lining

VALUE OF SOIL ARCH INDETERMINATE

It must be admitted that with time there has been in some cases an increase in the pressure on a tunnel lining, but the data available are not sufficient for the formulation of general rules. As a result, the mechanics of arch action are only imperfectly understood. So long as there continues to be uncertainty as to the ultimate durability of the soil arch, and as to the degree of protection afforded by it, the engineer is safer in discarding this arch as an active factor in sewer design. Ordinarily the maximum allowable loading on a sewer ring should be taken as no less than the normal pressure of the undisturbed soil.

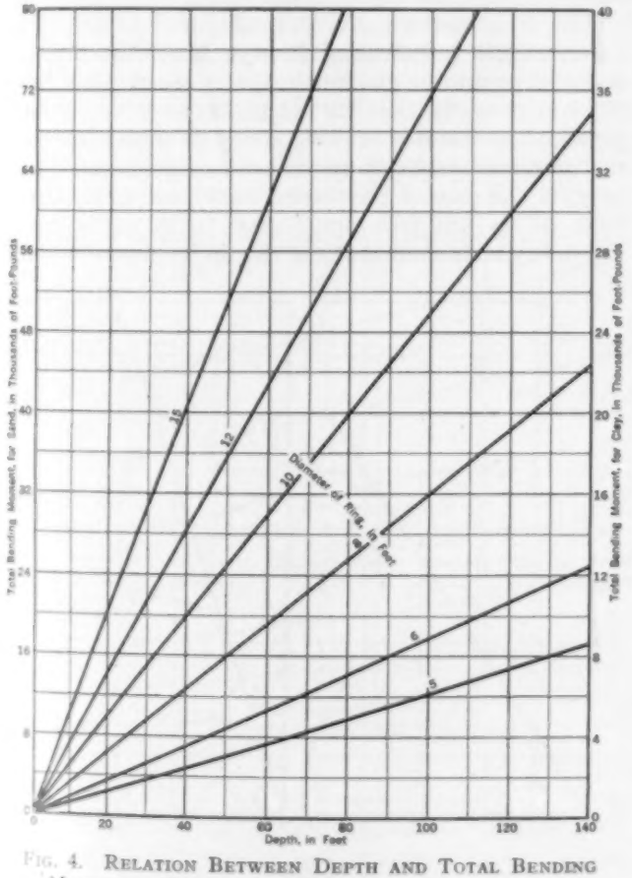


FIG. 4. RELATION BETWEEN DEPTH AND TOTAL BENDING MOMENT IN TUNNEL ARCH RINGS, FOR SAND AND CLAY

If a sewer is in an open cut, the pressure on the lining may exceed that produced by normal stress conditions, because of the slipping of the backfill along the surface of the excavated slope and the concentration of a considerable load at the bottom of the cut. In the case of tunnels, however, such a concentration of stress can only occur if the surface of rupture is within the soil mass, a condition that cannot exist unless there has been a previous ground movement. If the lining is sufficient to carry the normal loading, the initial slip cannot occur, and no pressure more severe than the normal can be built up. Accordingly, if no ground movements are allowed during construction, the normal stress condition is all that need be considered in the design of the lining for a tunneled sewer. In the special case of a cave-in during construction, it is suggested that the design for the particular section be altered to meet the pressures that would be expected in an open-cut sewer, for which design data have already been well worked out. If we are prepared to accept as the required loading a uniform vertical pressure equal to the weight of the superincumbent soil, and a uniform horizontal pressure equal to a given ratio of the vertical, then the determination of direct stress and bending moment is only a matter of applying familiar formulas. In Figs. 3, 4, and 7 this is done in the case of a monolithic concrete ring of uniform thickness. The vertical pressure, in pounds per square foot, is taken to be 120 times the depth of the soil, in feet, to the center of the ring, and the horizontal pressure is considered to be $^2/_3$ the vertical for rich clay, and $^1/_3$ for clean sand.

Experimental evidence of the value of ζ, the ratio of lateral to vertical soil pressure, has been accumulating for a number of years. Many of the tests used, however, failed to determine satisfactory values of ζ for various reasons. In some cases the vertical pressure was computed from the force applied to a piston, ignoring the fact that part of the total load was transferred by friction to the walls of the cylinder containing the soil sample; in others the pressure-recording device required lateral expansion of the soil, thus resulting in decreased pressure; and in still others, when clay was being tested, the load was increased beyond the bearing capacity of the sample without allowing for drainage, thus building up hydrostatic pressure. Some tests appear worthy of full confidence. Dr. Terzaghi has obtained the following values, measured by frictional resistance against a tape pulled through the soil:

TWO STAGES IN TUNNEL CONSTRUCTION
Needle Beams Support Ground Until Brick Arch Is Completed

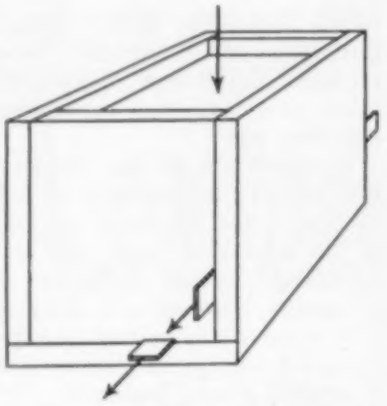


FIG. 5. DEVICE TO DETERMINE RATIO BETWEEN HORIZONTAL AND VERTICAL PRESSURES IN A SOIL SAMPLE

For pulverized quartz .... $\zeta = 0.42$
For yellow plastic loam ... $\zeta = 0.70$
For blue plastic clay ..... $\zeta = 0.75$

In an attempt to determine values of ζ for local soils, I have used a wooden box 6 in. square and 7 in. deep, as shown in Fig. 5. A thin soft brass sheath was laid horizontally on the floor of the box, parallel and close to one side. A strip of brass 1 in. wide was inserted in this sheath. A similar sheath and strip were placed flat against the adjoining wall, near the bottom. The pressure against the strips was taken to be proportional to the pull required to withdraw them from the sheaths. The box was filled with the soil sample to a depth of 6 in.; a wooden cover was placed over the soil; and pressure was applied with a jack. The container offered little resistance to the escape of excess water, and time was allowed for drainage before recording values. The results of the tests on a rather soft blue clay (a $1^5/_8$-in. steel ball dropped 12 in. on this clay made an indentation 1 in. wide) and a fine sand are shown in Fig. 6. For the clay, $\zeta = {}^2/_3$, and for the sand, $\zeta = {}^1/_3$.

In Fig. 7 the data supplied by Figs. 3 and 4 are used to compute arch thicknesses for concrete having a modulus of rupture of 500 lb. per sq. in. This corresponds to a compressive strength of about 3,000 lb. per sq. in. Of course these results are given for only two values of ζ, and the experiments were far too few to assure that these are limiting values. The value of ${}^2/_3$, especially, is probably often exceeded, but it is on the side of safety inasmuch as a value of unity produces only compressive stresses in the arch ring. In other ways also these calculations are approximate. In the case of shallow tunnels, for example, a possible saving in material might make it profitable to consider the difference between the pressure at the arch and that at the invert.

It should also be noted that although a tunnel in sand requires a much thicker lining than one in clay, at the same time it provides a considerably greater factor of safety, because the higher coefficient of internal friction of the sand increases the durability of the soil arch. Consequently, in tunneling through sand, if an engineer is confident that precautions taken in construction have eliminated cavities in the neighborhood of the sewer, he would be justified in using a ring thickness less than that required by the computed natural pressure. Similarly, in the case of clay, some confidence in the durability of the soil arch would seem to be admissible if laboratory tests show that the clay in its natural state is

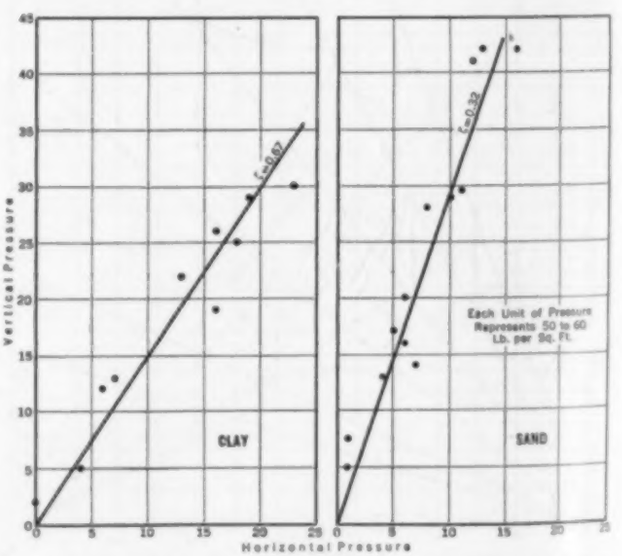


FIG. 6. RELATION BETWEEN HORIZONTAL AND VERTICAL SOIL PRESSURES, FROM TESTS BY THE AUTHOR

not appreciably compressed by a considerable increase in pressure. This might be true of clay in a deep stratum, which had been partly relieved of its overburden during centuries of surface erosion. However, the allowable reduction in ring thickness is so clearly dependent upon the unaided judgment of the engineer that such reduction should not be hazarded except after long experience with the soil under consideration. Moreover, the wall thickness as determined on the basis advocated in this article will generally be less than that found in existing sewers, and there would seldom be reason for further reductions.

For example, a tunnel 8 ft. in diameter, in clay at an average depth of 30 ft., requires a maximum wall thickness of only 7 in. according to Figs. 3, 4, and 7. Even in the case of no external pressure, few engineers would wish to decrease greatly the thickness of this 7-in. wall for an 8-ft. cylinder. Unless we are designing tunnels at far greater depths, therefore, there would appear to be little demand for the highly technical laboratory and mathematical analyses needed to justify a less conservative strength requirement for tunnel linings than that graphically expressed in the diagrams.

For purposes of illustration I have considered a sewer tunnel as a monolithic concrete ring. If such an assumption is used in actual design, considerable care must be taken to see that it is fulfilled. Longitudinal joints, when necessary, should be placed where the bending moment is zero, and should be suitably bonded. In placing concrete by hand, the tendency toward poor workmanship at the key must be carefully guarded against. Care must also be taken to tamp the concrete sufficiently at the key and in the invert—the points of maximum bending moment—so as to avoid the honeycomb structure that may result from the use of too dry a mix.

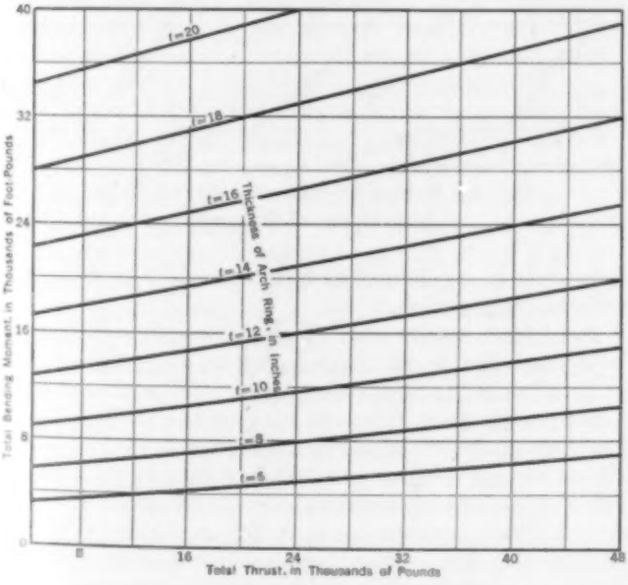
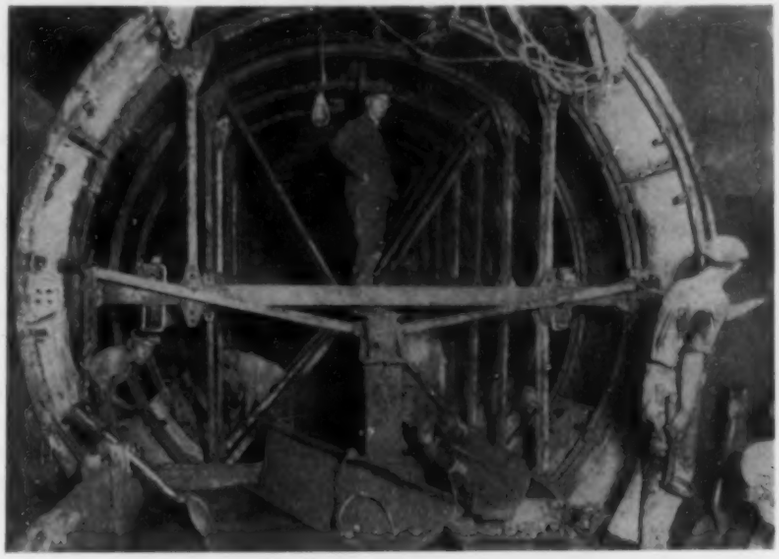


FIG. 7. ARCH-RING THICKNESS FOR CONCRETE
= 3,000 Lb. per Sq. In., and Modulus of Rupture = 500 Lb. per Sq. In.

Only circular sections have been considered here, but other sections may be readily designed by applying the same soil pressures and computing the thrust and the bending moment by the principles of static equilibrium and by the elastic energy equation.



USE OF STEEL FORMS FOR CONCRETE TUNNEL LINING
Ground Supported by Needle Beams Until Lining Is in Place

In this discussion of tunneled sewers it has been assumed that the soil is homogeneous and in its natural state. Frequently this condition is not fulfilled. For example, if a sewer is in soft clay overlain by a considerable depth of less plastic material, it is entirely possible that a more severe system of loading will develop. This occurs when the tunneling withdraws support from a section of the stiff stratum, so that arch action in the stiff material carries a considerable thrust to the soft material on each side of the tunnel and causes a greater "squeeze" against the sides of the tunnel than the pressure against the crown. To determine the resulting bending moment in such a case is beyond the limits of our present knowledge of soil mechanics. When the sewer is in sand or stiff clay overlain by soft clay, then the earth pressure at rest is greater than the initial pressure, and the method of design here shown can be used.

The use of steel at crown and invert, and elsewhere, is a means of reducing the required wall thickness and involves no special difficulty in applying the principles already developed. The justification of reinforcing on economic grounds can only be established by the circumstances of the individual case.

This method of design of tunneled sewers permits the engineer to select his own degree of accuracy and, to a large extent, his own factor of safety. Future research in soil mechanics will undoubtedly fix more closely the extent and durability of arch action in soils and so secure a more scientific approach to the ultimate solution. Meanwhile a rational discussion of the problem will serve to emphasize the oft-neglected distinction between open-cut and tunneled sewers and between different types of subsoil. It provides a standard for the design of the unusual case and a basis for the comparison of conflicting opinions.

Uncertainties of Hoover Dam

Closing Comments on the Adequacy of the Design

By M. H. Gerry, Jr.

Member American Society of Civil Engineers
Consulting Engineer, San Francisco

THE subject of Hoover Dam has been before readers of Civil Engineering *since October 1930, when Dr. Elwood Mead, Commissioner of Reclamation, described the project as a whole. His paper was thereafter discussed by other members. Subsequently, in the July 1931 issue, Mr. Gerry raised some questions about the design which were of such nature that space was provided in the August 1931 issue for rejoinder by representatives of the Bureau of Reclamation; and since that time other comments have appeared. Available space has limited this closing discussion to brief references to the more important points brought out. The present article brings the current papers on Hoover Dam to a close.*

In the August issue of Civil Engineering, Dr. Elwood Mead and Prof. H.W. Westergaard, Members Am. Soc. C.E., representing the Bureau of Reclamation, state that the Hoover Dam, as planned, is not a gravity dam but an arch dam, and further that a gravity dam is not rational for the Black Canyon site. For many years the Reclamation Bureau maintained that the only proper design for this site was a curved gravity dam, and as such it recommended to Congress essentially the same section as is now proposed, but which was then declared to be a gravity dam.

In compliance with a resolution of Congress, the Secretary of the Interior early in 1928 appointed the Colorado River Board, which on November 24, 1928, in its official report stated:

> The dam proposed by the Bureau of Reclamation and assumed to be the one referred to in H. R. 5773, is of the gravity type, curved in plan, with allowable stresses as high as 40 tons per square foot....It is the opinion of the Board that a dam of the gravity type is suitable for the site in question, and that such a dam built across Black Canyon would be safe, provided the maximum stresses allowed do not exceed those adopted in standard practice. (Report of Colorado River Board, House Document No. 446—70th Congress, Second Session, 1929.)

Dr. Mead and Professor Westergaard now declare: "The Hoover Dam will be essentially an arch dam.... Hoover Dam, as designed, is an arch dam in an arch dam site....Hoover Dam has been designed as an arch dam....It is not rational to put a gravity dam in this arch dam site."

On comparing the above statements, marked inconsistencies will be observed. If a gravity dam "is not rational" then the Reclamation Bureau was in error three years ago in recommending such a dam, and the Colorado River Board must likewise have been in error in declaring that a gravity dam was suitable for the Black Canyon site.

Public records disclose also this clear description of the structure as originally recommended:

> The unprecedented height of the proposed dam, roughly twice that of any dam heretofore built, necessitates a conservative design. The concrete gravity dam, built of homogeneous material and depending only on its weight to resist water pressure, rather than the uncertain stresses of thick arches, is best suited to this condition....No satisfactory solution was found for either a single or a multiple-arch dam for use at the lower site in Black Canyon. (Colorado River Development, Senate Document No. 186—70th Congress, Second Session, 1929.)

This was the situation in relation to the design at the end of 1928, the time of the passage of the "Boulder Canyon Project Act," authorizing the work now under way. So far as I know, the first public announcement from an official source declaring the structure for Black Canyon to be an arch dam is in these articles by Dr. Mead and Professor Westergaard.

Hoover Dam is not properly an arch dam but it is distinctly a gravity section, curved in plan, possessing all the structural characteristics and weaknesses of such a form of construction. Designers will not recognize the section as a rational one for an arch dam but will readily admit that it is a common gravity section.

Although Dr. Mead maintains that Hoover Dam has been designed as an arch dam, he states in support of his claims for safety, that the criterion of Maurice Levy for gravity dams has been complied with. Unfortunately for this argument, the criterion is not even approximately fulfilled when the curvature of the dam is taken into consideration, as it should be in treating gravity sections; and the deficiency will be far greater if earthquake shock is brought in, as claimed to have been done by those advocating this structure.

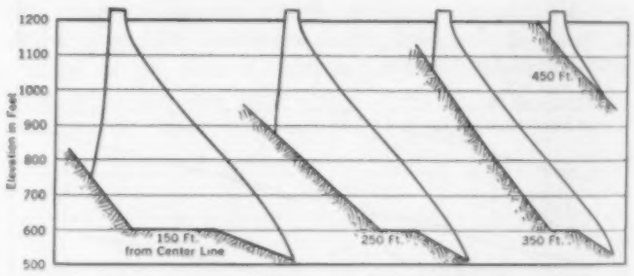


Fig. 1. Radial Sections of Hoover Dam
Nevada Side of Structure

Dr. Mead asserts that Hoover Dam is safe against every conceivable contingency, but nowhere have I seen proof of such broad claims. Elsewhere have been published maximum sections and plan views showing Hoover Dam as made up of vertical columns. In Fig. 1 are shown certain sections to a 500-ft. radius at the axis, located at the indicated intervals on the Nevada side of the river. These sections are scaled from the drawings accompanying the specifications, and disclose the impossibility of anything like uniform arch action throughout this dam. It is essential also in this case to consider the influence of the widely extended contact with the canyon

walls. Toward the end of the dam, radial sections all show steep downstream slopes under the base and very sharp tangential angles at the abutments. Under such conditions, arch action unless restrained may cause the breaking of the contact with the rock, thus endangering the dam.

Quite contrary to Professor Westergaard's statement, I strongly advocate structural testing, but hold that conclusions of value must be based on a great many tests, analyzed with the greatest care; and especially so in this case where the scale is very small and where in other respects it is impossible even approximately to represent conditions likely to prevail. The few tests made in the past on models of other dams indicate little more than the elastic relations developed under the special conditions of the tests, which were not at all those of actual dams in place. To base conclusions involving safety on the behavior of a 36-in. plaster-of-paris model, as evidence of the probable action of a 730-ft. dam containing about 3,500,000 cu. yd. of water-soaked concrete, certainly exceeds the limit of conservative engineering.

INTERRELATION OF UPLIFT AND SLIDING

The amount of uplift, as used in my paper, was first suggested by the late James B. Francis, Past-President Am. Soc. C.E. Many able hydraulic engineers have sustained this opinion, and some distinguished members of the profession have gone far beyond the recommendation of Mr. Francis and have insisted that for important structures an allowance equal to full head acting on the total area of the base be provided. The justification for this last assumption, which is twice as much as I used, rests of course on the importance of providing a wide margin of safety for great dams.

Dr. Mead and others representing the Bureau have attempted to disparage the plain intent and meaning of the criterion for uplift suggested by Francis. Certainly no engineer believes that an unbroken film of water extends under a dam, or that uplift necessarily varies uniformly from heel to toe, or that under-pressure is always distributed uniformly along the base; but variations from such conditions do not detract from the sanity of assuming a reasonable average figure, such as that used in my paper, and endorsed by many of the most conservative engineers of the day. Because this figure happens to be one-third greater than the assumption advocated by Dr. Mead is hardly a sufficient reason for its rejection, especially as it is decidedly more conservative.

The value of the sliding factor given in my article was an illustration for certain conditions. Dr. Mead doubts the correctness of this figure but he apparently misunderstands the assumptions and disregards important elements controlling the uplift. The value as stated is correct when based on concrete of 150 lb. per cu. ft.; but with gravel taken from the Hemenway Wash the reported weight of concrete is only 143 lb., which will increase the factor. The major uncertainty in determining sliding in all cases is the uplift. It is necessary in this case to allow for the fact that the base is sharply inclined downstream over a large part of its area and in the direction of the flow of the percolating water. The head producing uplift is not that acting at the point of water entrance under the dam but must be taken to the point of free discharge. This allowance results in a far larger uplift than that estimated by Dr. Mead.

It is important, also, to take into consideration that this dam is made up of vertical columns, thus introducing cleavage planes at about 50-ft. intervals in all parts of the structure. It is proposed to grout the joints between columns, but there will always exist fissures impossible of closure. When water under head is applied to a face of permeable concrete or to a fissured mass, it does not exert pressure alone on the surface but throughout the entire body, in proportion to the drop of the hydraulic gradient. It has been stated on eminent authority that hydrostatic pressure can be transmitted through seams measuring as little as $^1/_{500,000}$ in. The fine openings on the faces of the columns represent an enormous area where water pressure may act, resulting in a horizontal water loading distributed through the mass.

In further evidence of undesirable sliding conditions, factors are here given for a unit section at elevation 900; assuming a horizontal base and full downward pressure on the upstream face. The sliding factor for a curved section with a 600-ft. radius is 0.99; and for a straight section, 0.88. These factors are intended to indicate only the general situation, as a strict interpretation would involve consideration of the sloping bases. Space will not permit illustrations at other elevations, but it is apparent that this dam does not show satisfactory results when analyzed for separate gravity sections.

COMBINED ARCH AND GRAVITY ACTION STILL UNCERTAIN

Dr. Mead expresses great confidence in the "trial-load method of analysis"; a process founded on an elementary theory involving the possibility of determining equal deflections under load for a hypothetical arch rib and a hypothetical cantilever rib. Only for cases of thin arches of moderate height and of considerable length between abutments is it possible to obtain consistent results by this method. For a structure of the relative proportions of Hoover Dam, where the above conditions are not even approximately complied with, it is futile thus to seek a solution. Many uncertainties, such as the great variability of the modulus of elasticity, the plastic behavior of rock and concrete, temperature, water soaking, and earthquake shock, all combine to render valid conclusions unattainable by the trial-load method.

In reviewing statements by the advocates of the proposed structure, it is remarkable to note that, while all admit the existence of a very large number of uncertainties connected with every phase of the design, still the general assertion is made that the structure is safe under all conceivable conditions. Among the many uncertainties thus ignored, perhaps the greatest of all, for it represents a summation of many other uncertainties, is that surrounding the supposed dual support, thought to be derived from a combination of arch and gravity action. This vital uncertainty has never been solved, and no constructive technical suggestion has been offered in the present discussion. The outstanding disclosures of this discussion are: that Hoover Dam is now declared by the Reclamation Bureau to be an arch dam, although in form and appearance it is a curved gravity dam; that no technical analysis of Hoover Dam as an arch dam has been presented here or published elsewhere; and that sponsors of Hoover Dam have not yet proved its margin of safety.

HINTS THAT HELP

Today's Expedient—Tomorrow's Rule

The minutiae of everyday experience comprise a store of knowledge upon which we depend for growth as individuals and as a profession. This department, designed to contain practical or ingenious suggestions from young and old alike, should afford general pleasure not unmixed with profit.

Design of Flange Plates for Deck-Plate Girders

By C. M. GOODRICH

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THERE are many deck-girder spans in use today the strength of which is less than was intended merely because the designer estimated the lengths of cover plates according to the conventional parabolic formula and added two or three feet without intelligent consideration of the actual effect of this approximate method. The moment determined for the center of the span may fail to give the proper rating value for the deck-girder span under review. It may be wise from time to time to review customary methods and note their limitations.

Assume that a deck-girder span 90 ft. 0 in. long from center to center of bearings is to be rated by some E-loading. The following comparative moments at 10-ft. intervals from the end, and at the center, are for one rail, with E-40 loading. The moments from wheel loads in the first column were taken direct from a table; those in the second column have been calculated from the center moment by assuming a parabolic variation.

DISTANCE FROM END OF GIRDER In Feet	MOMENTS FROM WHEEL LOADS In Foot-Kips	MOMENTS BY PARABOLA In Foot-Kips	ERROR In Per Cent
Center	2,670.4	2,670.4	...
40	2,634.7	2,637.6	...
30	2,388.8	2,373.8	0.6
20	1,923.8	1,846.2	4.0
10	1,134.2	1,055.0	7.0

Flange-plate lengths are usually calculated on the assumption that the moment curve is a parabola. In Fig. 1, in which the parabola shows the area required

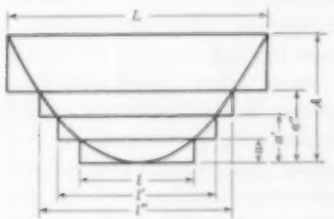

FIG. 1.

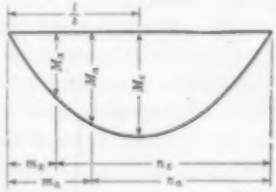

FIG. 2.

(by assumption) to correspond to the moment at the center, L is the span, A is the total flange area, and a, a', and a'' are the successive partial areas provided by the three flange plates assumed to be required, while l, l', and l'' are the three theoretical lengths to be determined. From the parabola we know that $\frac{l^2}{L^2} = \frac{a}{A}$ and hence we may write $l = \sqrt{\frac{a}{A}}\, L^2$. To determine these lengths, it is easiest to use a Mannheim slide rule. Opposite L on the D-scale set A on the B-scale, and opposite any a on the B-scale read the corresponding length sought on the D-scale. It only remains to add the right length to the results just obtained and the problem is solved. The uncertainty is as to how much should be added. The moment curve is not a parabola; it bulges at the end and at the quarter point. In an old German textbook, there appeared the suggestion that the parabola should be split at the center by a distance equal to half the distance between two of the main drivers. This approximation gives results which are fairly close to the true conditions at the points where the lengths of the flange plates are affected.

Investigate the amount of the error in the lengths of the flange plates derived from the parabola formula at the points where the moments are already known. It is of course assumed that the difference due to the dead load is relatively small; in designing it would be taken into account. Then find how far it is from the end of the girder to the point where the moment is 1,055.0 ft-kips instead of to the point where the moment is 1,134.2 ft-kips, assuming that the moment curve is a true parabola in the one or two feet between these two points.

In Fig. 2 is shown a moment curve. From the parabola, $M_t : M_x = \left(\frac{l}{2}\right)^2 : m_x n_x$.

$$\text{So } M_x : M_a = m_x n_x : m_a n_a \quad [1]$$

$$M_x = \frac{m_x n_x}{m_a n_a} M_a \quad [2]$$

$$\text{Also } m_x n_x = \frac{M_x}{M_a} m_a n_a \quad [3]$$

Using Formula 3 determine how far from the 10-ft. point the moment 1,055.0 ft-kips really lies.

$$m_x n_x = \frac{1{,}055.0}{1{,}134.2} \times 10 \times 80 = 744$$

$$9.2 \times 80.8 = 743$$

This shows that the error due to the parabola assumption is 0.8 ft., or 1.6 ft. in the length of a plate ending here. Similarly, calculations at the 20-ft. point show the error due to the parabola assumption to be 1.1 ft., or 2.2 ft. in the length of a plate ending at this point.

Figure 3 shows the moment curve, the parabola, and the curve derived by the German method. The moment

curve is for total moment and is drawn using moments derived from the wheel loads for 5-ft. intervals. The parabola is derived from the center moment. The outer curve is obtained, as stated before, by splitting the parabola at the center by a distance equal to half the distance between two of the main drivers.

DEPTH OF GIRDER CHANGES

There is another error in the usual method of determining cover-plate lengths, that is, the assumption that

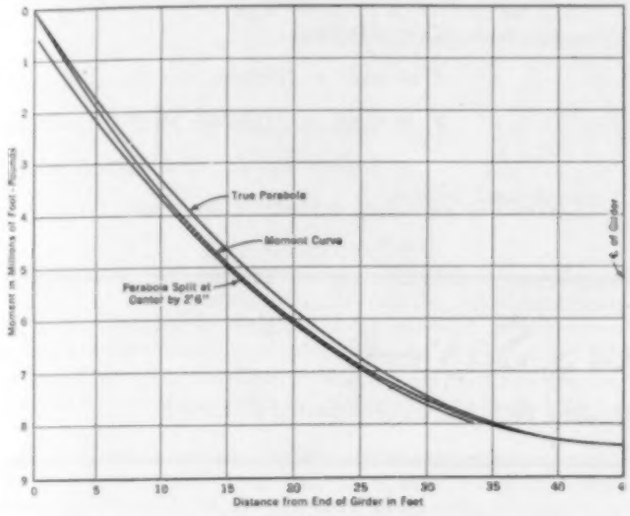

FIG. 3. MOMENT CURVES FOR A DECK-PLATE GIRDER BRIDGE For E-60 Loading on a 90-Ft. Span

the depth of the girder does not change. Assume the following make-up for a deck-girder flange for girders 90 ft. long between bearings and 9 ft. $0^1/_2$ in. deep over angles:

MAKE-UP OF GIRDER Dimensions In Inches	LENGTH OF COVER PLATES From Parabola In Feet	SECTION AREA in Square Inches	
		Gross	Net
2 angles 8 × 8 × $^3/_4$	..	22.88	18.38
$^1/_2$ of web area $\left(108 \times \frac{7}{16}\right)$	..	5.90	5.90
1 plate 18 × $^5/_8$	66.9	11.25	10.00
1 plate 18 × $^5/_8$	54.6	11.25	10.00
1 plate 18 × $^5/_8$	38.6	11.25	10.00
Total		62.53	54.28

The effective depth of the girder is: without cover plates, 8.66 ft.; with one cover, 8.79 ft.; with two covers, 8.90 ft.; and at the center, 8.99 ft. Using the usual assumption that the effective depth does not change, the errors are: with no covers, 3.7 per cent; with one cover, 2.2 per cent; and with two covers, 1 per cent.

If in the case just described the designer made the first cover plate 70 ft. long, the facts that the live-load moment obtained from the parabola is 7 per cent too small and that the effective depth is 3.7 per cent too great, would combine so that the girder might well be approximately 10 per cent weaker than a computer using more accurate methods would determine it. Also note that the more excess there is in the area provided at the center over the area required, the more nearly wedge-shaped the parabola becomes and the greater the error at the haunch where the first cover plates are added.

Volumes of Elliptical Heads for Tanks in a Horizontal Position

By GORDON J. ROBERTSON
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PITTSBURGH, PA.

IN AN EFFORT to simplify the calculation of the volume of liquid in the elliptical heads of a partially filled cylindrical tank, lying horizontally, I have developed a general formula and a table of coefficients. Similar tables for the ordinary dished head are already available.

Elliptical heads are generated by rotating an ellipse with a major semi-axis, a, and a minor semi-axis, b,

TABLE I. FACTORS FOR VOLUME OF ELLIPTICAL HEADS

PER CENT OF RADIUS	VALUE OF K	PER CENT OF RADIUS	VALUE OF K
1	0.000050	51	0.107942
2	0.000199	52	0.111765
3	0.000446	53	0.115637
4	0.000789	54	0.119556
5	0.001229	55	0.123521
6	0.001764	56	0.127531
7	0.002393	57	0.131585
8	0.003115	58	0.135681
9	0.003929	59	0.139820
10	0.004833	60	0.144000
11	0.005828	61	0.148220
12	0.006912	62	0.152479
13	0.008084	63	0.156776
14	0.009343	64	0.161109
15	0.010688	65	0.165479
16	0.012117	66	0.169884
17	0.013631	67	0.174323
18	0.015228	68	0.178795
19	0.016907	69	0.183299
20	0.018666	70	0.187800
21	0.020507	71	0.192398
22	0.022425	72	0.196992
23	0.024422	73	0.201614
24	0.026496	74	0.206263
25	0.028646	75	0.210938
26	0.030871	76	0.215637
27	0.033170	77	0.220361
28	0.035541	78	0.225108
29	0.037985	79	0.229877
30	0.040500	80	0.234667
31	0.043085	81	0.239477
32	0.045739	82	0.244305
33	0.048461	83	0.249152
34	0.051249	84	0.254016
35	0.054104	85	0.258896
36	0.057024	86	0.263791
37	0.060008	87	0.268700
38	0.063055	88	0.273621
39	0.066164	89	0.278555
40	0.069333	90	0.283500
41	0.072563	91	0.288455
42	0.075852	92	0.293419
43	0.079199	93	0.298391
44	0.082603	94	0.303369
45	0.086063	95	0.308354
46	0.089577	96	0.313344
47	0.093146	97	0.318338
48	0.096768	98	0.323335
49	0.100442	99	0.328334
50	0.104167	100	0.333333

about the axis b. The problem is to calculate the volume of the head when partially filled.

The volume of a full head may be expressed by the equation: $V = \frac{2}{3}\pi a^2 b$. By calculus, it is found that the volume of a horizontal slice of one head included between a horizontal plane passing through the axis of

the tank and a second plane at a distance, x, below or above the center of the tank, may be expressed by the equation: $V = \frac{\pi b}{6a}(3a^2x - x^3)$. The cross sectional area, A, of the tank may be expressed as $A = \pi a^2$; and the volume of a slice of the head may be expressed as $V = K_1 bA$, where $K_1 = \frac{V}{bA} = \frac{1}{6a^3}(3a^2x - x^3)$. Letting $x = \frac{ra}{100}$, where r = the per cent of radius of the tank,

$$K_1 = \frac{1}{6a^3}\left(3a^2\,\frac{ra}{100} - \frac{r^3\ a^3}{1{,}000{,}000}\right) = \frac{30{,}000\ r - r^3}{6{,}000{,}000}$$

When $r = 100$, $K_1 = 0.3333$, and $V = \frac{1}{3}bA$

The value of K_1 may be determined for values of r from 1 to 100, thus giving a table from which may be calculated the volume of any part of the head. For convenience, however, to give the volume from the bottom of the tank up to any horizontal plane, the value of K has been tabulated instead, in Table I, where $K = 1/3 - K_1$.

For example, a tank with an inside diameter of 8 ft. has an elliptical head with a rise inside of 2 ft. What is the volume in one head when the liquid is 12 in. deep?

$a = 4$, $b = 2$, and $r = \frac{12}{48} \times 100 = 25$ per cent.

From the table, $K = 0.028646$.

$A = \pi x 4^2 = 50.2656$

$V = KbA = 0.028646 \times 2 \times 50.2656$

$= 2.8798$ cu. ft., the volume which it was desired to find.

Our Readers Say—

In Comment on Papers, Society Affairs, and Related Professional Interests

Need for Uniform Power Records

TO THE EDITOR: Those who, like myself, are engaged in judging the comparative performance of electric properties and in studying the economics of the industry, are in full accord with Dr. Smith's comments on the importance of facts and the difficulty of ascertaining them, published in his paper in the March issue. Although in the last decade accounting and statistical records have been improved in comparability and simplicity, there is still much to be done.

Certainly there is a need for greater uniformity of records. I wish that I could find systems of accounting and statistics which would permit a real comparison of the operating properties (located in nine states and, together, producing between 5 and 6 per cent of the electricity sold in this country) that form the group with which I am associated. One source of confusion is that, although properties in any one state usually keep records on substantially the same basis, municipally owned plants in certain states are not under regulation. This confusion increases when properties in different states are included in the comparison. The difficulty of obtaining comparable data on municipal plants results in part from the practical impossibility of separating the proprietary from the governmental functions of municipalities.

In our own group I have attempted to obtain a uniform treatment of two apparently very simple facts: (1) the number of kilowatt hours sold to the several classes of customers; and (2) the revenue realized from such sales. But this involves such questions as, what is a domestic customer; what is the distinction between industrial and commercial customers or between large power users and small power users; and where is the dividing line? There is also the problem of separate meters and separate rates for domestic cooking and refrigeration and for illumination and small appliances on the same premises. The uniform treatment of non-uniform conditions is as much a technical, statistical, or economic error as the non-uniform treatment of uniform conditions.

There are many requests for facts. Data on our group of properties required by the Federal Trade Commission filled 17,000 pages. Subsequently the Federal Power Commission asked for information which ought to have been easily obtainable from these pages. The two commissions, however, approached their problems differently, and there were technical, engineering, and accounting problems involved in searching original records and reassembling figures. Also data covering the same properties have been furnished to the U.S. Census Bureau, the Bureau of Mines, the Bureau of Internal Revenue, and to other Government boards and bureaus.

I should like to point out without criticism, that any attempt at unification of our accounts and statistics by any branch of the Federal Government should follow rather than precede unification of the practice of the several Government bureaus who are required to secure statistical and accounting information from the operating companies.

Aside from operating matters, I realize that the classification of physical property has not yet been completely worked out from the standpoint of cost analysis. There is more information available on some properties than on others. Also, there are geographical differences and differences in the type and extent of development of the areas served, all of which have a distinct bearing on the situation.

I am thoroughly in accord with Dr. Smith on the subject of the importance of well conceived and accurately assembled statistical data as the basis for sound judgment on the part of engineers, but it seems that the problem he so ably discusses is one to which there is no very simple solution. The facts we most certainly have, and yet it is frequently not only extremely difficult

to determine, record, and compare these facts but frequently very expensive, as the railroads have learned during the last fifteen years by the increase of clerks' compensation from 4.09 per cent to 6.52 per cent of revenues and by the increase in the number of these clerks by nearly 60,000, both conditions resulting largely from the increasingly detailed and elaborate reports required by the Interstate Commerce Commission. Perhaps, as engineers, we can best serve industry by making sure that an accurate functional analysis of business precedes the call upon business to bear the cost of accumulating the facts.

F. W. DOOLITTLE, M. Am. Soc. C.E.
Vice-President, The North American Co.

New York, N.Y.
March 2, 1932

Granite Composite Surface Finish for Concrete

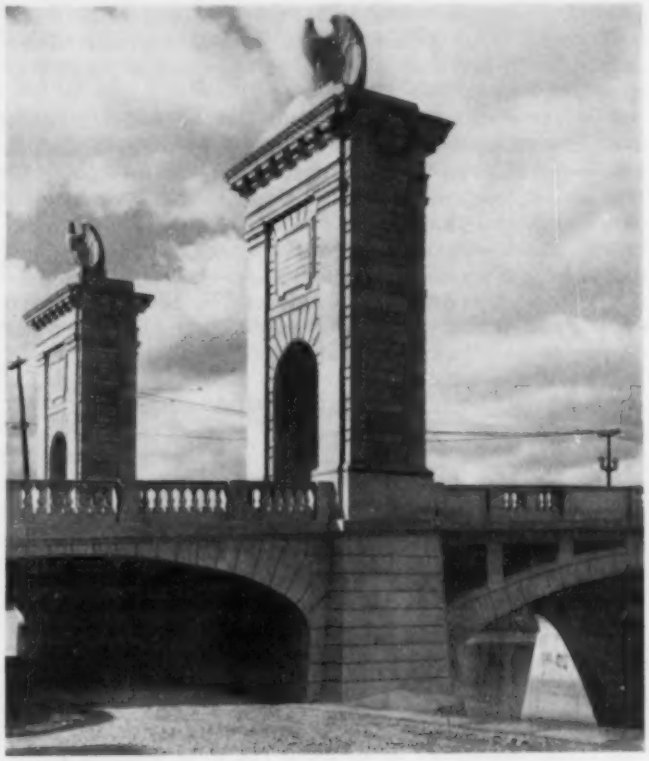
SIR: In connection with the article, "An American Memorial Bridge in Belgium," by Edward H. Tashjian in the January issue, it is interesting to note that practically the same method for producing an artificial granite surface was used successfully in this country from 1926 to 1929.

The Market Street Bridge across the Susquehanna River at Wilkes-Barre, Pa., is a reinforced concrete structure about 1,500 ft. long and 93 ft. wide, located in a beautiful park. Esthetically designed to conform with its surroundings, the bridge consists of twelve graceful elliptical arches with spans varying from 80 to 120 ft. The pylons and the eagles are of Indiana limestone, and the pier nosings, pylon bases, coping, and balustrades of New England granite, but the greater part of the surface of the bridge is faced with granite composite. These different types of finish are shown in the accompanying photograph of one end of the bridge.

After experimenting with the mix and type of material, it was decided to use granite screenings from the same quarry as the stone for the trim. The stone dust included everything passing a No. 4 sieve which was graded to give a dense mass. The mix consisted of 1 part of portland cement to 4 parts of this granite dust with sufficient water to make a workable mortar. The dry granite dust weighed about 92 lb. per cu. ft., measured loose before mixing, and the average requirement for the job was 34.4 lb. per sq. ft. of granite composite in place.

The method of installation was to place wire mesh the required 2 in. back from the face of the forms and pour the composite mortar in front of the mesh just ahead of the concrete mass in order to get a monolithic bond between the two. The $\frac{1}{2}$-in. square mesh was of 20-gage galvanized welded wire that was ordered in rolls 24 in. wide. On vertical faces the mesh was wired to the reinforcing steel, or where steel was not called for, $\frac{5}{8}$-in. round rods were set vertically 2 ft. 0 in. apart to stiffen the mesh. The sheets were placed horizontally so that the composite could be poured and puddled in 2-ft. lifts. On horizontal surfaces such as the soffit of the arch rings, the wire mesh was omitted.

Bush hammering was not started till the concrete had cured at least 28 days. The chipping was done with hand air hammers, for which special types of bits were designed for the different kinds of work. The bits that stood up best under the work were made of $1\frac{1}{4}$-in. sq. bars of deformed high carbon reinforcing steel tapered at the shank to fit the hammer and then upset at the end to $1\frac{1}{2}$-in. square. For plane surfaces and fast work the head was made with 25 teeth, ground pyramidal in shape to a depth of $\frac{1}{4}$ in. For finer work 36 teeth ground to a slope of about 60 deg. with a depth of $\frac{1}{4}$ in. were used successfully. A double chiseled bit ground to a slope of about 45 deg. with the face of the bit slightly rounded was found to be the best for chipping edges and corners.



MARKET STREET BRIDGE ACROSS THE SUSQUEHANNA RIVER AT WILKES-BARRE, PA.
Showing Granite Composite Finish on the Arches

For overhead work the air hammer was attached to a 2-in. pipe mounted on a tripod and counterweighted so that the operator was relieved of the weight of the hammer. The cross arm was adjustable so it could be clamped at the required height and swung. This meant that about 60 sq. ft. of surface could be bushed without moving the tripod. With this outfit an experienced worker would do about 30 sq. ft. of plane surface of bush hammering per hour.

From the construction standpoint, the application of the granite composite slows up the operation of pouring the concrete except in large masses and of course increases the unit cost, but produces a finish much superior to carborundum-rubbed concrete. A recent examination shows that the surface is weathering well after four or five years.

The Market Street Bridge was designed by the late B. H. Davis, M. Am. Soc. C.E., and after his death the work was carried on under the direction of David A. Keefe, M. Am. Soc. C.E. Carrere and Hastings were the architects, and I was chief engineer for Walter S. Rae, of Pittsburgh, who was the general contractor.

R. DE CHARMS, M. Am. Soc. C.E.

Rochester, N.Y.
February 15, 1932

Ventilation of Tunnels Costly

DEAR SIR: Mr. Frankland's excellent paper entitled "Tunnels or Bridges—Which?" in the February number, gives a thorough comparison of the relative costs of these two methods of crossing a navigable body of water. My experience confirms the conclusions that Mr. Frankland draws from his comparisons. Very roughly, a tunnel of the same capacity as a bridge will cost twice as much as the bridge. This proportion obviously varies with such local conditions as those cited in Mr. Frankland's paper. In the majority of cases a bridge is not only more economical than a tunnel, but it is also preferred by motorists on account of being in the open air and free from gas fumes.

The ventilation of the Holland Tunnel seems to be as complete and perfect as it is possible to make it, but it has been achieved at a very great expense for construction and upkeep of the ventilation plant. I agree with Mr. Frankland that the bridge is to be preferred to the tunnel unless local conditions of approaches and requirements for navigation clearances make the tunnel imperative. From an esthetic standpoint, a bridge and its approaches often lend themselves to pleasing architectural treatment and thus become an important consideration in beautifying a city.

RALPH MODJESKI, M. Am. Soc. C.E.
Consulting Engineer

New York, N.Y.
February 16, 1932

Tunnel Crossings Often Justified

TO THE EDITOR: I was interested in Mr. Frankland's article on bridges and tunnels in the February issue. Since vehicular tunnels are reasonably new they are justly subject to comparison with bridges. Personally I hold no brief either for bridges or tunnels, but expect to design and build whichever type offers the best solution for a particular crossing. From my own study of vehicular crossings in this country and abroad, I am convinced that tunnels will be used when special conditions make bridge costs abnormally high, where Government rulings prohibit bridges regardless of cost, or where it is important on account of the cost of real estate or on account of earnings to locate the portals near the water or near business centers of connected cities. The governing factors in the situation are given here briefly.

Governments, in the interests of commerce, occasionally demand high ship clearances which make the cost of bridges extraordinarily high. They also add to bridge costs by demanding unobstructed channels of great width necessitating long spans. The resultant long and high spans must have a proportionately greater width on account of the necessary stiffness against the wind, and such bridges are consequently built with more traffic lanes than the amount of traffic justifies. Also, great clearance heights necessitate long and expensive approaches. This involves considerable real estate and makes it impractical to locate the ends of approaches near the business centers of cities or close to the water.

In the case of the Detroit-Windsor Tunnel and the Ambassador Bridge, it would not have been practical to build a bridge at the same location as the tunnel whose portals are near the river and immediately adjacent to the business district. As a result of its central location the tunnel gets much more traffic than the bridge in spite of competition with an adjacent ferry. Since the two lanes of traffic in the tunnel are capable of accommodating five times the present traffic, it is impossible to concede any advantage to the bridge with its five traffic lanes. The City of Antwerp, Belgium, after years of consideration, is now building a vehicular tunnel under the Schelde. A high-level bridge would have necessitated locating the approaches too far from the water and the business center.

Terminal conditions may make it inexpedient to concentrate traffic at a single crossing on account of the difficulty of traffic distribution. Since the cost of a tunnel per foot is not much affected by its length, whereas the cost of a bridge increases more nearly as the square of the span and also increases with the height, it is obvious that tunnels will not generally be economical for narrow rivers but may be so for wide channels requiring long spans and high clearances. Also, tunnel crossings may be justified where the cost of the large and heavy bridge foundations is very high, the load of a tunnel upon its foundation being almost negligible.

The cost of building vehicular tunnels is rapidly decreasing because of improvement in design and methods of construction. At Detroit we used pre-cast tunnel sections in the river portion, decreasing both cost and time of construction. We also developed a structural steel lining for the shield-driven portions and found it much cheaper than the cast iron.

On a basis of these facts, I am of the opinion that tunnels offer no universal solution for river crossing problems, nevertheless they meet conditions in certain localities which justify their construction. I am, further, strongly of the opinion that it is unfair to compare the two types of structures in respect to cost per lane unless the lanes can be loaded with traffic in a reasonable length of time.

W. J. DOUGLAS, M. Am. Soc. C.E.
Parsons, Klapp, Brinckerhoff and Douglas

New York, N.Y.
February 24, 1932

Tunnels Suffer by Misleading Comparison

TO THE EDITOR: The paper in the February issue entitled "Tunnels or Bridges—Which?" contains numerous erroneous cost figures and misleading statements. The limited space available permits me to point out only a few of these.

On page 90 Mr. Frankland speaks of "the comparable cases of the Holland Tunnel and the Philadelphia-Camden Bridge," wholly ignoring the respective widths of waterway, 5,500 ft. for the tunnel and 2,600 ft. for the bridge, which evidence the futility of such a comparison. Another illustration of misleading figures is seen in Table I, which follows.

TABLE I. COSTS OF PHILADELPHIA-CAMDEN BRIDGE

	Total Cost	No. of Lanes	Cost per Lane
Author's estimate of 1917	$11,000,000	8	$1,375,000
Author's completed cost, in Table I	26,000,000	10	2,600,000
Actual cost, exclusive of interest during construction	37,065,000	6*	6,177,500
Actual cost, exclusive of interest during construction	39,500,000	6*	6,583,000

* The bridge has only six completed lanes, with provision for four future lanes or tracks.

Even if Mr. Frankland's fallacious cost per lane could be accepted, the actual cost per lane was $2^1/_2$ times the figure shown in his Table I, and $4^1/_2$ times his estimate of 1917. That the over-all lengths of this bridge and the Holland Tunnel were so nearly equal, despite the fact that the Hudson is more than twice as wide as the Dela-

ware, is largely due to the War Department's required clearances of 135 ft. for the bridge and 55 ft. for the tunnel.

Mr. Frankland's processes, using as many as 26 lanes per bridge, reduce his comparisons to impossibilities. While it is possible to deal on paper with that many lanes, I know of no bridge having more than six lanes which can be used to its full capacity, because of limitation of approach facilities. To anyone familiar with traffic conditions where a bridge of as many as 16 lanes could be even contemplated, the impracticability and prohibitive cost of adequate approach facilities would be generally apparent. Rarely indeed would a comparison of cost per lane, assuming more than 8 vehicular lanes for the bridge, have any value whatever. Furthermore, it is seldom that both vehicular and rail traffic can be centered upon one structure without serious sacrifice in operating conditions or approach costs. In fact, both types of traffic frequently must accept a bridge location dictated by topographic, real estate, and foundation considerations. A tunnel, however, can usually be located solely as required by traffic conditions.

The Holland Tunnel, with its four lanes, has handled a maximum daily traffic of 58,702, the Delaware Bridge, with six lanes, 64,667. The maximum hourly traffic per lane of about 1,300 is very nearly the same for both. If additional traffic lanes are provided on the bridge, very large sums of money must be spent for approach facilities in Philadelphia. Mr. Frankland states that the costs of these structures are reflected in the toll charges of 50 cents and 25 cents, respectively. Rather, these charges correspond closely with the charges of the nearby ferries established before the tunnel or bridge was constructed and reflect the logic of charging half the price for crossing a river less than half the width.

The paper omits any reference to the tremendous indirect cost of the blight which the long approaches of a high-level bridge frequently impose upon the real estate values of the extensive surrounding territory, the "bridge basement." This effect is well illustrated by the Brooklyn, Manhattan, and Williamsburg bridge approaches, where real estate values during the last fifteen years have not even remained stationary but have actually declined, while the values in practically all other parts of New York City have advanced substantially. In contrast, the territory surrounding the New York approach of the Holland Tunnel has, since the building of the tunnel, been built up with modern structures and the land values have multiplied several times.

Again, Mr. Frankland compares the cost of the Holland Tunnel with that of the George Washington Bridge. Erroneous conclusions may be drawn, however, unless certain facts not mentioned in the article are borne in mind. The public authorities who built the tunnel were created in 1906 for the specific purpose of building a bridge across the Hudson. The traffic crossing the river was found to be centered downtown at a point where the tunnel was eventually built, but after some years of investigation and study the bridge project at that point was abandoned as economically not feasible, and a tunnel was therefore the only practicable solution. Incidentally the tunnel is a great financial success. The George Washington Bridge, on the other hand, is built at the only natural bridge site over the Hudson in the 12-mile length of Manhattan, and where the river is only 3,500 ft. wide, with bedrock coming to the surface at the river's edge on both sides, and the banks rising abruptly to high elevations.

OLE SINGSTAD, M. Am. Soc. C.E.

New York, N.Y.
March 7, 1932

Bridges Less Costly Than Tunnels

TO THE EDITOR: In the February issue, Mr. Frankland's masterly and incontrovertible comparison of costs of bridges and tunnels in general is most interesting. Included in my paper, "Bridge versus Tunnel for the Proposed Hudson River Crossing at New York City," in Vol. 84 of TRANSACTIONS is a diagram comparing the costs of tunnels with those of suspension bridges having main spans varying from 1,500 to 3,000 ft. From it was drawn the conclusion that, for the greater limit, in the case of highway bridges without railroad tracks, the ratio of costs of tunnel and bridge was about 1.5 for equal carrying capacity. This ratio is much smaller than that found by Mr. Frankland. The fact is accounted for by two valid reasons:

1. Estimates on highway tunnel costs given me by tunnel experts were based on preliminary estimates, and were afterward found to be far too small.

2. At the time the paper was written, studies of tunnel ventilation had not been completed, and neither the cost of its installation nor that of its annual operation had been determined. The actual figures for these costs are so great that, when all annual expenses for operation and fixed charges are capitalized, the total first cost for the tunnel is very largely increased.

The experiment on the feasibility of ventilating highway tunnels has been completed, and the results are fairly satisfactory. My forecast of the excessively great expense, however, has been fully borne out. No developments in the past decade have given me cause to reverse my conclusion that, for North River crossings, highway traffic is much more economically carried by bridges, and railway traffic (always to be electrified) by single-track tunnels.

An interesting addition to Mr. Frankland's general comparison may be made by referring to my paper, "Quantities of Materials and Costs per Square Foot of Floor for Highway and Electric-Railway Long-Span Suspension Bridges," in TRANSACTIONS, Vol. 91. In it there is a detailed cost estimate for the bridge of 7,000-ft. span and its approaches, having 18 lanes of traffic, amounting to almost $300,000,000 or $16,700,000 per lane. The corresponding estimated cost per lane for tunnels, as given by Mr. Frankland for the Narrows, or Staten Island-to-Brooklyn, Tunnel, is $19,500,000; but the length in this case is 500 ft. greater. Reducing the tunnel cost correspondingly, the comparable figure will be $18,500,000, which shows that, even for this enormous span-length, the tunnels for equal carrying capacity would still be more expensive than the bridges.

J. A. L. WADDELL, M. Am. Soc. C.E.
Waddell and Hardesty, Consulting Engineers

New York, N.Y.
February 20, 1932

Erroneous Comparisons

TO THE EDITOR: Aside from the fact that the relative merits of tunnels and bridges cannot be determined by comparisons in which important factors are ignored, I would like to take issue with a few of the incorrect statements made by Mr. Frankland in the February number.

The two two-lane tunnels comprising the Holland Tunnel have lengths of 8,554 and 8,369 ft., respectively, between portals, and are about 9,250 ft. from end to end, including approaches. But Mr. Frankland, stating that

the length of the proposed 57th St. Bridge is 10,000 ft. or "about the same" as that of the tunnel, proceeds to estimate the cost of eight such tunnels, using lengths of 17,410 ft., or nearly twice the actual length. He compiles the cost of these eight tunnels, providing 16 traffic lanes, as 9,500 ft. at $3,000 per ft., plus 7,910 ft. at $1,500 per ft. which, multiplied by 8, gives his total of $323,000,000. In another place he states the cost of the twin Holland tunnels as $55,000,000, which would make four such tunnels, providing 16 lanes, cost $220,000,000—a trifling difference of $103,000,000!

He speaks of the proposed Mid-Town Hudson Tunnel and, in his Table I, compares it with bridges on the basis of costs per lane; but he does not give the length of this project nor mention that it not only crosses the river but the Palisades as well, involving, in addition to the river crossing, a length of perhaps two miles for the portion wholly west of the river.

In quoting from his report upon the Delaware River crossing, Mr. Frankland speaks of "... the danger of asphyxiation which always exists in an automobile tunnel in spite of all precautions..." As a matter of fact, the Blackwall Tunnel in London has been in safe operation for about 30 years, and other European tunnels nearly as long. During its four years' use the Holland Tunnel has carried nearly 50,000,000 vehicles without a ventilation failure of even minor importance.

According to Mr. Frankland the traffic capacity of bridges is "considerably greater" than that of tunnels, ostensibly because the strict spacing of vehicles in the tunnels is not necessary on bridges. The fact is that this control in the tunnels is inspired by the somewhat obvious purpose of securing maximum capacity, together with the briefest possible time of crossing safely; and it accomplishes both of these purposes.

Quoting again from the article, "For a tunnel the annual cost of operation would be at least $2,000,000 more than the comparatively small sum required for a bridge"; but in another place the annual charge for operation and maintenance of the Holland Tunnel is given at $1,500,000. From this it follows that, far from involving any expense, the operating personnel and public service companies would pay the bridge half a million dollars a year for the privilege of working and furnishing electric current, supplies, and other necessities.

In comparing the Ambassador Bridge with the Detroit-Windsor Tunnel, the article gives the cost of the latter as $25,000,000, or over $7,000,000 more than the actual cost of $17,468,234, including the interest during construction. My figures were published in *Engineering* for Nov. 14, 1930, with the sanction of one of the members of the engineering firm responsible for the construction. Even if Mr. Frankland's highly exaggerated costs could be accepted, his contention that the economic values of these two structures are in the ratio of 1:2.77 is hardly borne out by the fact that the two lanes of the tunnel are used to nowhere near their capacity, while the five lanes of the bridge carry even less traffic than the two of the tunnel.

No doubt in many instances study from an economic viewpoint would show a bridge to be preferable to a tunnel, but where a special structure as a vehicular tunnel is to be considered, all the facts should be taken into account. The respective locations, if not identical, must be studied in relation to traffic demands. If more lanes are assumed than are warranted by the traffic, their economic value must be reduced. In any case, if a larger number of traffic lanes is justified, the cost of fully developed approaches which will adequately feed the bridge without undue congestion of the city streets, must be included. Since the existing concentration of traffic in many of our city streets is a matter of serious concern, it is evident that the suggested dumping of 12 or 16 more lanes in any one area is not to be lightly accepted.

CHARLES E. SUDLER, M. Am. Soc. C.E.

Bronxville, N.Y.
March 5, 1932

Bridges Superior to Tunnels

DEAR SIR: In his excellent article, in the February issue, Mr. Frankland presents data of very great value both to engineers and public officials. My personal experience of the past three years confirms Mr. Frankland's findings on the subject of bridges and tunnels.

As consulting engineers for a certain county, my firm helped secure a War Department permit for a bridge to cost about $3,500,000. The application to the War Department was for a high-level bridge across a river approximately one mile wide, and of a maximum depth of about 30 ft. with comparatively flat, low, undeveloped, and inexpensive land upon either side for the construction of the approaches. The application was bitterly fought by navigation interests, who were represented by a respected and generally well informed engineer. At the War Department public hearing, this engineer made the remarkable statement that, at this particular location, a vehicular tunnel could be constructed more cheaply than the proposed bridge. This statement was presented without either substantiation or qualification.

To controvert it, we presented our experience with a crossing for the Oakland Inner Harbor, between Oakland and Alameda, Calif. Several years ago, complete bridge plans were prepared for a structure, containing a 200-ft. double-leaf bascule span and providing for a 36-ft. roadway, with two street car tracks, one 10-ft. walk, and two railroad tracks. Sometime after the preparation of these plans, a vehicular tunnel was constructed instead of the bridge. The cost of this tunnel amounted to more than twice the estimated cost of the bridge, and the tunnel of course provided only a part of the facilities which had been planned for the bridge.

In concluding my answer to the engineer who represented the navigation interests, I presented the statement that, under ordinary conditions, a vehicular tunnel could be expected to cost twice as much as a bridge, which would provide at least twice the traffic capacity of the tunnel. This statement is rather well borne out by Mr. Frankland's detailed and thorough comparisons. Incidentally, the War Department issued a permit for the building of the bridge.

The Holland Tunnel, with its ventilation system, is a remarkable engineering and construction achievement. It appears fair to state, however, that there are only a very few communities in the world which could afford such a tunnel. Likewise, there are few communities where conditions require a tunnel to the extent that one is required along the lower portions of the Hudson River. Two factors which may make the adoption of a tunnel imperative are those of navigation and the advanced state of development of the land on each shore. Also, there are a few waterways over which the War Department will not permit the building of any bridge, as the destruction of the bridge, either purposely or accidentally, would hamper the use of an important port. The Narrows and the lower Hudson River at New York are cases in point. For such locations the tunnel is the only solution.

With respect to land conditions, there are cases where the building of a bridge with long approaches is likewise out of question. As an obvious illustration, the Wall Street section of New York City could not possibly be expected to serve either as a bridge plaza or approach. In lower Manhattan therefore, several well distributed tunnels would undoubtedly afford more direct and convenient traffic routes for most of the users than would one or two large bridges.

If, however, there is any section of the United States other than lower Manhattan, which has conditions warranting the use of a vehicular river tunnel, or which will provide sufficient traffic to furnish proper financial support for such a tunnel, I have not heard of it.

E. R. NEEDLES, M. Am. Soc. C.E.
Ash-Howard-Needles and Tammen

New York, N.Y.
February 29, 1932

Type of River Crossing Depends Upon Location

TO THE EDITOR: The article by F. H. Frankland, "Tunnels or Bridges—Which?" in the February issue, is based on the most recent information on the subject and helps to further clarify a question about which there has been considerable discussion and speculation from time to time.

So far as crossings for vehicular traffic mentioned in the article are concerned, the general conclusion that the cost of tunnels per traffic lane is from $2^1/_2$ to 4 times the cost of bridges appears indisputable. For an entirely equitable comparison, however, some of the cost figures given in the article would probably involve considerable modification, because they undoubtedly reflect the personal and time factors involved in the design, in the cost estimates and experienced costs, and in the method of financing. Also, they probably reflect material differences in the extent and character of the approaches. But the comparison of cost of construction and cost of operation per lane of traffic represents only one factor in a general economic comparison of tunnels or bridges, or in a comparison of these two types for any specific crossing. The total first cost of comparative projects, irrespective of capacity, may be a more important and, possibly, a compelling factor.

In the case of any important highway crossing, the cost of new highway arteries which may have to be provided to connect with the crossing may even outbalance the cost of the crossing proper. Also the comparative cost of vehicular operation over the crossing proper as well as over existing or new connecting arteries must be taken into consideration. The effect of a crossing and its connecting arteries upon the public welfare, upon property values, and the economic effect upon existing crossings may have an important influence upon the selection of the type of crossing. In view of the complexity of the question it is hardly feasible to arrive at reliable conclusions, except on the basis of careful and equitable comparative study in the case of each project.

In his article, Mr. Frankland makes the statement, "A long-span bridge requires a certain minimum width for lateral stability. With a span of over 3,000 ft., as in the George Washington Bridge, the width should be not less than 120 ft." In my opinion this often quoted empirical rule, which is justified in the case of smaller bridges, should not be applied to long spans such as that of the George Washington Bridge, or even to spans of considerably lesser length.

We have been forced to modify our views concerning the rigidity of long-span bridges. The George Washington Bridge, as designed and built, is ample proof that the relative vertical and lateral rigidity, as required in smaller spans, is not necessary. This bridge has a very flexible lateral system, the rigidity being supplied largely by the enormous dead weight of the suspended structure, the cables, and the heavy floor. It would have been perfectly feasible and permissible, without impairing the lateral rigidity, to make the suspended structure much narrower, as would have been sufficient for from 4 to 6-lane vehicular traffic. The essential fact to be kept in mind is that a bridge of this size can be built for a larger than minimum capacity at comparatively small additional expense, but structural reasons do not require that it be built for this larger capacity. A fair economic comparison between a tunnel and a bridge crossing should take this fact into consideration.

If there appears to be no justification for building a crossing for more than four lanes of vehicular traffic, then the comparison should be based on this capacity for both types of crossing. In the case of the George Washington Bridge, careful comparative estimates indicated that if the bridge had been built for a capacity of from 4 to 6 lanes of vehicular traffic and 2 footwalks only, a saving of from $15,000,000 to $20,000,000 could have been effected. It appeared wise, however, to invest this additional amount in order that the capacity might be increased when needed and justified in the future.

O. H. AMMANN, M. Am. Soc. C.E.
Chief Engineer, The Port of New York Authority

New York, N.Y.
February 15, 1932

Value of Steel with High Elastic Limit

TO THE EDITOR: Professor Thompson's refreshing account, in the February number, of M. Freyssinet's work on long-span arch bridges shows France still a generation ahead in the art of reinforced concrete construction.

The main principles advocated in this paper are: (1) concrete of high strength hooped with a large percentage of high carbon wire; (2) a minimum amount of longitudinal reinforcing; (3) box sections of arches; (4) the smallest possible section at the crown and a great multiple of this section at the springing line; (5) long-span longitudinal girders; (6) hollow abutments of large size; (7) scientific design at the centering; and (8) successive strengthening of the main arch members to make them self-supporting at the earliest moment, thereby allowing the centering to be made much lighter.

All these principles have been thoroughly discussed and advocated in engineering literature throughout the world since the beginning of this century, which by no means detracts from M. Freyssinet's credit in insisting that only their widest application makes long-span bridges of reinforced concrete practically feasible. A prism strength for plain concrete of from 15,000 to 20,000 lb. per sq. in., after 90 or even 365 days, seems about 100 per cent too high even if very rich mixtures and cement guns or vibrators are used. In a 5-ft. cylinder with a spiral of $^3/_4$-in. wire at a $^3/_4$-in pitch, the spiral represents only 4 per cent of the reinforcing; and with special steel

having a yield point of over 200,000 lb. per sq. in., the increase of compressive strength of the core would be only 16,000 lb. per sq. in. Even if several concentric rings of spirals were used it would be difficult to increase the spiral reinforcing to more than 6 per cent. Besides, tests on such inner spirals are still lacking. It would be comparatively easy to use from 20 to 30 per cent of longitudinal reinforcing in the shape of structural members having a yield point of possibly 100,000 lb. per sq. in. which would bring the maximum compressive strength of the composite section up to 62,000 lb. per sq. in. This is a very respectable capacity but still is not as far-fetched as M. Freyssinet's 70,000 lb. per sq. in. without the 30 per cent of longitudinal reinforcing.

Box sections of arched bridges, with small crown and large springing sections with hollow abutments, have been used by M. Hennebique even in his earliest bridges. A notable example is the 100-meter bridge over the Tiber at Rome, built in 1910. Quite a number of such bridges up to 500-ft. span appeared in my *Reinforced Concrete Pocketbook*, published in 1909. Dr. F. Emperger gave a theoretical discussion of such designs, and especially of the favorable influence of large springing sections, in his book, *Neuere Bogenbrucken*, published in Berlin in 1913, in which he also proposed successive strengthening of the arch ribs in order to lighten the centering.

Long-span longitudinal girders are a great advance over the comparatively short spans used in this country. In bridges having a rise of one-fifth or even one-eighth of the span, where small crown sections are used, the so-called parasitical stresses amount to only a few per cent, which are more easily taken care of in the design than by M. Freyssinet's method. Stresses of 1,800 lb. per sq. in. in the form work are certainly not new; the tables on form work in my book are computed for these stresses.

For American practice I prefer to build first a steel arch representing the longitudinal reinforcing, strong enough to support its own weight and that of the form work and of the first stage of strengthening the main arch members by hooped concrete. This is more economical than M. Freyssinet's method of building enormous centerings and not using any longitudinal reinforcing.

L. J. MENSCH, M. Am. Soc. C.E.
General Contractor and Constructor

Chicago, Ill.
March 7, 1932

Eliminating Highway Billboards in Ontario

DEAR SIR: In recent issues of CIVIL ENGINEERING, several contributors have condemned the practice of placing billboards along the highways. Undoubtedly a considerable number of persons in all parts of the country agree with this feeling. That the situation is not completely hopeless is evidenced by the fact that an editorial in the columns of the *San Diego Union* for December 1, 1931, calls attention to vastly improved conditions in Ontario, Canada. In this province a law has been passed which gives the highway department authority to destroy objectionable billboards and charge the costs to the property owners concerned. The only signs permitted are those conforming to the standards of the highway department and advertising a business conducted on the property on which the sign is located.

I have been opposing for many years the indiscriminate advertising that has developed along our roads, and have protested against the authorities allowing (as they, particularly the courts, have done) the growth and establishment of a sort of "vested interest" now difficult to weed out. For example, in my book, *Highway Location and Surveying*, the thought was expressed that: "...commercial advertising along the highways should by law be restricted to business done on the premises, and...then the signs themselves should be subject to the approval of the highway authorities. Such an exercise of the police power of the state would be in the interest of public welfare."

I think that the highway authorities of Ontario are to be congratulated on their stand in this matter and that they are deserving of all the support that we can give them.

W. W. CROSBY, M. Am. Soc. C.E.
Consulting Engineer

Coronado, Calif.
March 1, 1932

Stiffening Columns Against Buckling

DEAR SIR: I should like to comment further on the design of columns loaded between the supported ends as mentioned by Mr. Downey in his letter in the September issue.

The proper treatment of many of these special problems will be greatly clarified by reference to the following books: *Columns* by Ernest H. Salmon; *Applied Elasticity* by Stephen Timoshenko and John M. Lessels; and *Applied Elasticity* by John Prescott. The first book treats columns fairly fully. Together with the reports of the Society's Committee on Steel Column Research and those of the U.S. Bureau of Standards, the first book gives a good basis for understanding the various phases of column behavior. The second book contains many examples of the use of approximate methods of solving buckling problems by the theory of least work, using the Fourier series of one or two terms. The third volume deals with many buckling problems from the more exact mathematical standpoint, as well as by the method of approximation.

It may be noted that Timoshenko's solution of the pin-ended Euler strut carrying a load at the center, by the method of least work, differs somewhat from the Vetter formula given on page 45 of your January issue. For webs of columns and for outstanding legs or flanges of columns, Timoshenko also gives buckling stresses which check remarkably well with the tests of the Society's committee. His plate-girder web formulas, however, being based on assumptions regarding boundary conditions not found in plate girders, are distinctly misleading. The work of Prescott on thin tubes axially loaded checks extraordinarily well, even in the case of thin paper cylinders, such as are described in Technical Note No. 370 of the National Advisory Committee on Aeronautics. Many other practical problems may be evaluated, provided the actual conditions are substantially met by the theoretical assumptions.

I think it can be shown that there is not much chance of serious error in cases such as that proposed by Mr. Downey if the column is designed for an end load of 350,000 lb. and the lower 12 ft. of it are strengthened with plates to bring the area up to that required for 500,000 lb. In lieu of the foregoing design, two sections of the same roll number may be used—one designed for the 20-ft. length, the other for the 12-ft. length—making sure that the light section about the axis of greatest radius of gyration is suited to the 32-ft. length.

Now a 12-in. H 92-lb. column, 20 ft. long carries 364,000 lb., without exceeding a slenderness ratio of 120.

A 12-in H 120-lb. section will carry 530,000 lb. as a 12-ft. column. The radius of gyration of the former is 5.40 in., and the slenderness ratio (based on a 32-ft. length and the greatest radius of gyration) is 71.1. The allowable stress is 14,053 lb. per sq. in. The total area of the smaller section is 27.06 sq. in., and the total load is 380,300 lb. Hence, with the web restrained, the 92-lb. section can carry a 350,000-lb. load both on an unsupported length of 20 ft. and on a length of 32 ft. unsupported in the plane of its major axis.

Although a pin-ended strut formula is far from reasonable for columns having a slenderness ratio as small as, or smaller than, 100, by using Timoshenko's coefficients it will be found that, for a load applied at the center (which is a worse case than that given by Mr. Downey) it is necessary in calculating the Euler critical load to use a length about 0.2 per cent greater than 32 ft. and compute the maximum load as applied to the 120-lb. section Now the maximum radius for the 120-lb. section is 5.51; the slenderness ratio is 69.7; the area of the section, 35.31 sq. in.; and the allowable stress, 14,174 lb. per sq. in. Hence the maximum load is 500,000 lb. Perhaps if allowance is made for the 0.2 per cent increase in length, a 126-lb. section should be used. It would have been possible to use 10-in. H sections if necessary.

For members much longer than those needed in this case, the effect of the correction in length might be more important. However, it is probable that the pin-ended strut method is somewhat severe in its demands. It must be remembered that the use of two sections, such as those just calculated, assumes a very rigid joining by welding. I should not recommend a riveted joint at such a location. If welding is impracticable, plates should be added to the 92-lb section where required.

There is need for a much more fundamental understanding of buckling problems, as the design of light structures must inevitably increase the number of such problems.

EDWARD ADAMS RICHARDSON

Bethlehem, Pa.
February 2, 1932

Wrong Conception of a Principle of Elasticity

SIR: There seems to be a misconception concerning a certain relation between the deformations of an elastic system and the forces that cause such deformations. As this idea is rather widespread, both in America and abroad, I should like to bring the question before the profession briefly.

Let Fig. 1*a* be an arch fixed at both ends and assumed, for simplicity, to be symmetrical. Imagine that the end B is free and that an unknown force is applied to it by means of a mechanical contrivance, so that the point B must be displaced in an assigned direction, say, vertically. Then if we measure this displacement, δ_1, and the vertical displacement, δ_2, of a point C, we can write, according to the prevalent theory:

$$\text{Vertical reaction at } B = \frac{\delta_2}{\delta_1} \times (\text{the force applied at } C). \quad [1]$$

From this equation the designer is supposed to be able to compute his vertical reaction at B.

All this is fundamentally wrong. A well known theorem, by Guidi, will demonstrate the inaccuracy, not only of the whole conception, but of Equation 1 as well.

In Fig. 1*b*, AB is the same arch as in Fig. 1*a*. Imagine an unknown force applied at B so as to displace the free end B. Let $\Delta's$ be the horizontal projection of the variation of the span s, due to the displacement of B; H', the variation of the horizontal thrust, due to this deformation; and δ' the vertical displacement of any point, C.

Now imagine (Fig. 1*c*) that the end B is hinged and on rollers, and that a vertical force of unity is applied at C. Call the corresponding variations of the chord and the horizontal thrust for this new condition of equilibrium Δs and H, respectively. Then, by the general principle of reciprocity, we can write:

$$1 \times \delta' = H' \times \Delta s$$

But we can also write:

$$H' : H = \Delta's : \Delta s$$

Therefore:

$$\delta' = \left(\frac{H}{1}\right) \Delta's$$

which is the form that Guidi gave to this theorem.

From the preceding equation we can obtain the relation

$$H = \left(\frac{\delta'}{\Delta's}\right) \times 1. \quad \ldots\ldots\ldots \quad [2]$$

Now, let us apply the same principle to the vertical component, $\Delta'y$, of this displacement (Fig. 1*b*). By removing the rollers at B and by exactly the same reasoning, we obtain:

$$V = \left(\frac{\delta_1}{\Delta'y}\right) \times 1. \quad \ldots\ldots\ldots \quad [3]$$

In their outlines, both Equations 2 and 3 resemble Equation 1, but they are far less inclusive than Equation 1.

In fact, the movements δ_1 and δ_2 in Equation 1 do not represent free displacements, for the end B has been forced along a certain arbitrary path and its rotation has been fully restrained. In so doing a vertical force has been applied at B; but in addition a horizontal force as well as a moment would have to be applied, neither of which has been taken into consideration.

In Equations 2 and 3, on the contrary, the quantities, δ', δ_1, $\Delta's$, and $\Delta'y$ are the result of a free deformation

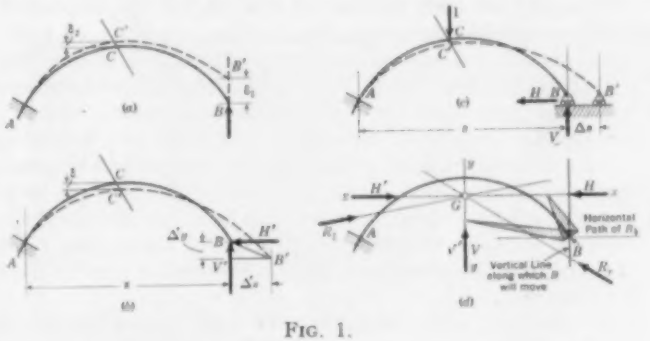


FIG. 1.

and—this is the most important point—they relate to one hypothesis of loading, while H and V are the components of the reaction depending upon an entirely different hypothesis of loading, namely, force 1 acting at C. In Equation 1 all this strictly theoretical truth has been confused so as to make a mathematical inconsistency.

The ellipse of elasticity shows the error of this theory in a still more glaring way. Let Fig. 1*d* be a symmetrical arch; G, its elastic center; and xx and yy, the two axes of its ellipse. The vertical displacement of the free end, B, without any rotation, can be caused only by a vertical

force, V, rigidly connected with B, and acting along yy. Similarly, its horizontal displacement without rotation can be caused only by a force, H, as shown. Hence, the right reaction, R_r, of the arch will pass through G. By identical reasoning, if A is free and B fixed, the left reaction, R_l, having V' and H' as its components, will also pass through G. The positions of the reactions being thus determined, the computations of their moments with respect to the end sections would be useless.

ALFREDO C. JANNI, M. Am. Soc. C.E.
Consulting Engineer

New York, N.Y.
January 28, 1932

Soil Test Not Always of Value

TO THE EDITOR: I am glad that Mr. Yates, in his article in the March issue, confirms the opinion of most engineers that wash borings—those in which the samples consist of cuttings carried to the surface by water—have little value in determining the character of a soil. The blind faith in tests of this type that has existed for many years has always been inexplicable to me.

"Dry cartridge" samples can be obtained with light drilling rigs from all formations except heavy gravel, boulders, and ledge rock. If the foundation is to rest on such strata through direct bearing, as by caisson or cofferdam, samples of the strata can be obtained by means of a rotary core drill at only slightly greater cost than the bucketed samples obtained with the old fashioned heavy churn or well drill. The bucketed sample is scarcely more dependable than the washed sample.

Sufficient borings should be taken to discover any important variations that may exist in the subsoil and to determine their extent. Load tests, made in the field on soil or on piles, usually are used to check the engineer's judgment as to carrying capacity; but similar tests of rock are difficult and expensive. The bearing capacity of rock varies greatly with only slight differences in its appearance. This uncertainty has properly led to most conservative rock loadings. However, as the result of much study and test work, it is now possible, by means of compression tests conducted in a laboratory, to predict with reasonable accuracy the bearing capacity of rock. Such tests are made on unfractured samples, which may be taken from the cores of borings or from pieces removed by air tools or blasting. The horizontal plane should be clearly marked on the sample so that it may be tested in the same position with respect to the test load as the rock is loaded in the field by the structure. While fairly large samples are preferable, this test can be made satisfactorily on core samples as small as $1^3/_8$ in. in diameter. The thickness of the sample should be somewhat greater than its diameter.

For testing, a hole slightly over 1 in. square is cut into the top of the specimen to provide level bearing for the testing tool. The bottom of this hole is leveled carefully with plaster-of-paris. The specimen is next laterally confined in plaster-of-paris within a cylindrical steel container slightly larger than the specimen, and all voids around the specimen are filled with plaster-of-paris. After the plaster-of-paris has set, the steel container and specimen are placed in a testing machine, and the load is applied through a tool 1 in. square resting on the bottom of the hole in the top of the specimen. The load is increased, first, to the yield point, and then to the point of ultimate strength.

In the softer or more elastic rocks, the yield point often is only half of the ultimate strength. In the harder rocks, there is often no yield until near the point of complete failure, at which point all rocks break down into small grains. Since the load at the yield point and at the point of failure is quite uniform for various specimens of the same rock, I believe it is possible to establish a safe unit bearing value for rock by applying a definite factor of safety to its ultimate strength as determined by this test, just as is done with other structural materials.

Tests of this nature have been made in a limited way for the last five years. They indicate that the safe bearing capacity of rock can be assumed conservatively to be at least one-half the load at the yield point in this test. This is considerably higher than the ordinary loads now allowed on rock, and corresponds on the average to a factor of safety of four on the load at the point of ultimate strength.

Unusually high loadings on rock are used in the placing of open-end steel piles; 72 tons or more per square foot is often allowed in point bearing under conditions not permitting of adequate inspection. Engineers frequently approve such a loading when they would hesitate to allow 25 tons on the same rock under caissons, where the material can be carefully cleaned and adequately inspected.

It is believed that, as a result of the greater confidence that further laboratory tests will give, higher rock loadings in excavations will soon be universally recognized.

J. W. TAUSSIG, M. Am. Soc. C.E.
Vice-President, Raymond Concrete Pile Co.

New York, N.Y.
March 3, 1932

Turkish Water Works Device

TO THE EDITOR: In the January issue on page 33 there is presented material on a Turkish water works device called *suterazisi*. The structure shown on the right of the page is located near, rather than on, the Aqueduct of Valens. It is of comparatively recent origin and does not date back to the time of Emperor Valens—as is intimated.

This photograph represents a different type of structure from that represented in the photograph to the left, in that its function was that of dividing proportionately the incoming stream of water. In order to accomplish this, an open divided basin of stone or lead was placed on the top of the obelisk. The incoming stream was conveyed by a lead or clay pipe to the largest of the compartments. Holes of various sizes permitted the water to flow from this compartment into each of the others, thus dividing the stream into a number of smaller ones in a proportion determined approximately by the areas of the holes, assuming uniform outlet conditions for the several basins. Pipes conveyed the proportionately divided streams to the various consumers, providing at a given point a continuous, approximately uniform supply. To meet the fluctuations in demand, each consumer or each public fountain was provided with a small reservoir or "cistern."

This device still finds a limited use in Constantinople (Istanbul) today. Its origin seems uncertain, but it is quite certain that it was used in Byzantine times, and possibly also as far back as Roman times.

WALDO E. SMITH, Jun. Am. Soc. C.E.
Assistant Professor of Civil Engineering
North Dakota State College

Fargo, N.D.
February 11, 1932

Soil Pressures on Subways Analyzed

TO THE EDITOR: In connection with Mr. White's article in the March issue, the following suggestions are offered. A rational computation of the pressure distribution across a flexible foundation is possible only on the assumption that the soil is truly elastic—that is, that the resistance offered to vertical compression is in direct proportion to the settlement at any point. To what extent this assumption holds true for various types of soil is still a matter for future research.

The elasticity of a soil is usually expressed by the soil pressure constant, representing the load which, when applied to 1 sq. ft. of foundation, will cause it to settle 1 in. Its value varies widely. Whereas Professor Föppl found it to be 200,000 lb. for virgin garden soil, the Chicago Subway Commission uses values as low as 500 lb. for clay.

In an elastic soil, the flexure line of a foundation, since it defines the settlement, must at the same time be a graph of the pressure distribution. This is expressed mathematically through a differential equation, the solution of which is too complicated for general use. As a result of this fact, the economy attainable by correct design is often sacrificed. In other indeterminate structures, this condition has been improved in recent years through model analysis by means of which intricate problems are solved in a fraction of the time required for mathematical treatment. The saving in time and mental effort thus effected has no doubt strongly stimulated the use of indeterminate structures.

That model analysis can also be advantageously applied to soil pressure investigation is indicated in Figs. 1 and 2. Models of this type have been used by the Board of Transportation as far back as 1927, and the results obtained have been incorporated tentatively in a set of rules insuring increased economy in invert design wherever the character of the soil is definitely known. The immediate purpose in this case is not, as in ordinary model analysis, to determine the influence lines, but simply to obtain a graph of the flexure line from which the soil pressure and moment at any point can be easily found.

In these models the invert is represented by an elastic wire, the soil pressure by tension springs. The springs are calibrated, all of them showing the same elongation under the same pull, and are so arranged and adjusted that the wire, when not acted upon by other forces, will assume the position indicated by a straight line some distance above it.

The loads from columns and walls are proportionately represented by weights attached by strings either directly to the wire or to cantilevers soldered to it, thereby permitting loads to be applied eccentrically for the purpose of taking care of moments. Thus the moment at the bottom of the sidewall is reproduced by attaching the corresponding weight to the left-hand cantilever at a point to the right of the wall axis a distance equal to the moment divided by the wall load. When applied, the weights will cause the wire to be displaced downward until equilibrium is established between the pull of the weights and springs and the stiffness of the wire. The latter will assume a deflected shape which, in a correctly proportioned model, is a true graph of the pressure distribution across the invert.

The models shown represent only one-half of the respective structures, as both are symmetrical about the center line. Since in such cases a tangent to the flexure line at the point of symmetry must be horizontal, the unknown moment at that point is found by applying a weight corresponding to one-half that of the center column load to the right-hand cantilever, adjusting the amount of eccentricity until, when all the weights are applied, the free end of the wire projecting beyond the point of symmetry is displaced parallel to itself.

In Fig. 1 is shown the model of a standard Board of Transportation invert for a two-track subway, and in Fig. 2 that of a proposed design for a four-track station of the Chicago Subway. The assumed soil pressure constants are 12,000 and 500 lb., respectively. In the Chicago model the sidewall is represented by an extra wire since, as it has about the same stiffness as the invert, it exerts considerable restraint. In the Board of Transportation design, however, the stiffness of the sidewall is negligible and need not be considered in the model.

ANDERS BULL, M. Am. Soc. C.E.
Assistant Engineer, Board of Transportation of the City of New York
New York, N.Y.
March 2, 1932

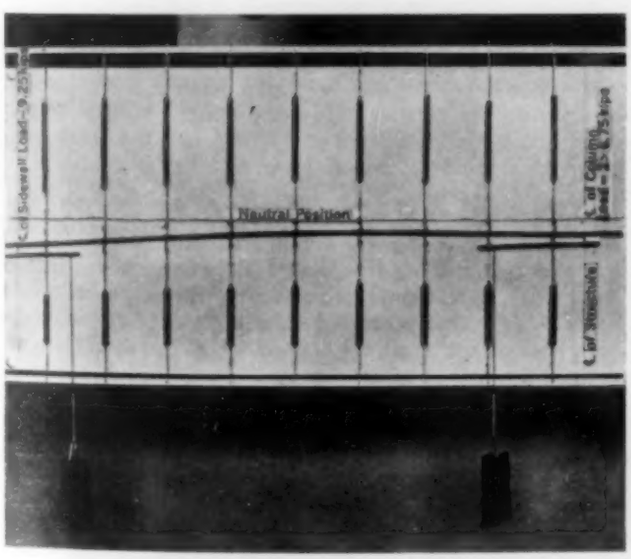


FIG. 1. A TWO-TRACK NEW YORK SUBWAY

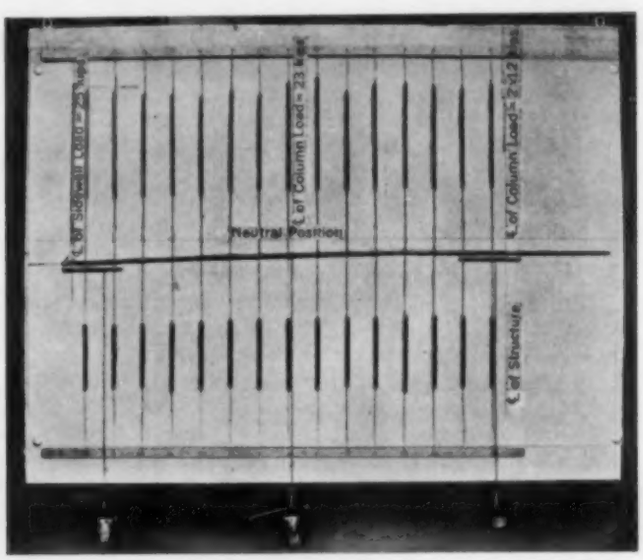


FIG. 2. A FOUR-TRACK CHICAGO SUBWAY STATION

Soil Pressure Distribution Models

Accuracy Important in Surveys

DEAR SIR: Mr. Syme's article on "Geodetic Control for North Carolina Highways," in the March issue, gives a clear and forceful exposition of a widely prevalent weakness in surveying, and provides an effective step toward a remedy.

The accuracy with which the geodetic monuments, described by Mr. Syme, will be set is of prime importance. It would appear that the minimum degree of accuracy required for surveys to be incorporated into the system should not be less than 1 in 5,000, corresponding to the U.S. Coast and Geodetic third order of traverse, as these will form the basis for setting new monuments. Even this will require careful work in the mountainous sections of the state, if the instruments and methods of chaining employed in the usual surveys are used.

It should be observed that this proposed monumentation will be coordinated by latitude and longitude, whereas local surveys will be coordinated by plane coordinates in feet. Surveys where the plane coordinates from one center of reference meet those from the adjoining center of reference would require adjustment along the line of division. If belts 150 miles in width are used, a correction angle of approximately $1^1/_2$ deg. would be introduced at the line of division. This could be avoided on a given survey by referring it entirely to the nearest established center. It may be found advantageous not to adopt a specific series of plane systems for the state. Few local surveys of areas will extend more than 25 miles. In fact, the only surveys likely to be of considerable length are those for county boundaries, railroad lines, and highways. These may be tied to the control monuments at intervals and coordinated from one monument to the next. The bearings for an east and west line would require an adjustment of about half a degree at 50-mile intervals.

Surveyors will note with satisfaction that an azimuth line will be established at each control monument. This should win the cordial approval of those who have recollections of rising in the small hours of a succession of cold winter nights for the purpose of ascertaining the position of Polaris, and finding the star obscured by clouds.

The suggested central organization would appear to be quite necessary if full benefit is to be derived from the proposed scheme. In addition to the duties outlined by Mr. Syme, the organization should keep the scheme before the public by occasionally issuing, to engineers and surveyors, maps and bulletins showing the progress made. The experience to be gained by such an organization and by the State Highway Commission in tying in its own surveys, will afford a basis for an equitable and acceptable set of rules to govern surveys. And this should bring about the formulation of laws which will require that sufficient information be shown on all maps and surveys filed in the county courts, so that they may be reproduced on the ground at any time, and by any surveyor.

The proposed monuments, being set at intervals of about 50 miles, would require, in the maximum case, a traverse of about 25 miles to tie a survey to the nearest monument, and if such a traverse be closed, for checking, the total tie-in survey would be 50 miles. This would introduce an appreciable item of expense not justifiable in small surveys. The difficulty will tend to disappear with the accumulation of tied-in surveys and additional monuments. In surveys of magnitude, the extra expense will be inconsiderable, both as compared with the total cost of the survey, and because the distance to the nearest monument will be less than 25 miles.

An attractive feature of the scheme is the low initial cost to the state desiring such geodetic monumentation, due primarily to the efficiency of the U.S. Coast and Geodetic Survey; and there is little doubt that the increasing necessity for accurate surveys will induce other states to follow the lead of North Carolina—particularly those states not subdivided into townships.

O. B. BESTOR, M. Am. Soc. C.E.
Civil Engineer

Baltimore, Md.
March 3, 1932

Need for Development of Latin America

TO THE EDITOR: It occurs to me in connection with Dr. Waddell's letter in the November issue that, with prices on existing bonds at their present low state and with interest payments suspended on many of the issues, it would be impossible, at this time, to interest American investors in new issues. Even if there were an immediate revival of demand for foreign bonds, it is extremely doubtful whether it would be possible to market additional securities at any time in the near future because of the present chaotic condition of the foreign securities market.

In the meantime, it is highly desirable that the potentially rich Latin-American countries continue to develop. From their standpoint, continued development means renewed and enlarged payrolls, modern and more economical means of producing their products, and quicker and cheaper means of transporting those products, not only to their own centers of consumption but to the outside world as well.

From our standpoint, increased buying power in Latin-American republics would prove a most beneficial step toward recovery of our lost national prosperity. A stable and flourishing foreign trade is the most profitable warehouse in which to store the surpluses that occur under a system of mass production. In many cases, these surpluses represent the profits of the enterprise. Latin America is our logical field of foreign trade, and in our sister republics of the South there is an increasing tendency to consider the quality of imports as well as their price. If this is kept in mind American exporters will have a distinct advantage.

Further developments in Latin America will depend directly on the further influx of outside capital. In the future, the American investor will demand a more concrete guarantee, in his foreign investments, than a mere agreement to pay.

The plan suggested by Dr. Waddell for an "engineering-contracting-financing syndicate" provides the necessary factors for the renewed development of Latin America. It also provides a means whereby money can be raised for badly needed construction in those countries and, at the same time, guarantees the investor that the money will be spent wisely on worthwhile projects.

In normal times it is extremely doubtful if such an enterprise, no matter how ethical its character, could be launched in Latin America. However, with many of our Southern neighbors in their present condition of financial distress, there is little doubt that the syndicate, introduced by the well-conducted educational efforts of selected men and offering greatly needed capital, could gain a trial there. With a chance to demonstrate itself, its future continuation would rest entirely on the results

it could produce, both for the Latin-American countries involved and the investors here.

A. W. GREEN, JR., Assoc. M. Am. Soc. C.E.

New York, N.Y.
March 8, 1932

Latin Americans Lack Confidence in Foreign Proposals

TO THE EDITOR: The syndicate, suggested by Dr. Waddell in the November issue, for the purpose of making "a systematic and scientific effort to develop all of Latin America," ought to be very helpful, especially if it is founded on principles of impartiality, equality, and honesty, and provided the psychology of the Latin-American race is well understood from the beginning.

Statistics show that, in the past, hundreds of North American individuals and organizations have operated in Latin America, but I doubt whether many of them have been successful in these engineering enterprises. Sooner or later all of them have encountered some sort of difficulty and, consequently, have been unable to establish themselves in Latin America.

Personally, I would devote all my efforts to bringing about the realization of Dr. Waddell's proposed syndicate, but feel that there is an insurmountable difficulty that will prevent its establishment. This is the fear of Latin Americans for North American gold and the lack of understanding between the two races. The probe on foreign bonds carried out recently in Washington strengthens the first part of my statement so far as Latin America is concerned and shows clearly the truth of the second part of it. My authority for this is a South American engineer, who has written to several of his engineering friends for their opinion on the matter. Latin Americans would like to have plenty of money and cooperation from their colleagues in the North, but they fear strongly that, with this money, will come political complications and harmful interventions.

In the future, if money is loaned to Latin America, no matter from what source or for what purpose, it must be loaned at its own risk on account of the dangers of unstable governments and revolutions. If capital does not like the protection offered by the government of the country in which it is proposed to establish the "engineering-contracting-financing" enterprise, I suggest that the "potential resources and business possibilities" of the country be forgotten and that better and surer fields of exploitation be sought.

The engineering profession is supposed to be practical, and if Dr. Waddell can find a "practical" solution that will change our feelings and enable us to accept American gold without fear, he will live in our memories as a benefactor to both Americas. But if, owing to our financial difficulties we accept his suggestion for a syndicate in fear, the results will not be helpful.

RAFAEL A. HIDALGO, Assoc. M. Am. Soc. C.E.

Ancon, C.Z.
February 29, 1932

Should Latin Americans Be Members of Trade Syndicate?

TO THE EDITOR: A careful study of Dr. Waddell's plan for a Latin-American syndicate, as published in the November issue, shows that he has given a great deal of thought to this proposition. One salient point made by Dr. Waddell is that now is the proper time to lay the foundations for the suggested syndicate; and it seems to me that the proposal should be developed to such a stage that it will be ready to function as soon as conditions are favorable for undertaking the enterprises contemplated.

There is one question which apparently has not been considered either by Dr. Waddell or the engineers who have thus far commented upon his suggestion—and that is the advisability of making the proposed syndicate a mutual enterprise, in which the citizens of Latin America could become active members.

According to my observation, cooperation and confidence between citizens of our country and their Latin-American associates are necessary to the success of certain projects in which they are mutually interested. In many instances in the past, confidence has been practically destroyed and irritation has been aroused by the serious error made by many engineers and business men from the States in the assumption that we are superior to the inhabitants of most of the other parts of the world. This is far from being true. There are many Latin Americans who are the equal of the best of our own people and when they are treated with an air of superiority by those who are really their inferiors, there is a natural resentment reacting against North Americans in general.

The part of Dr. Waddell's plan which calls for lectures to be given throughout the Latin-American countries is most excellent. One of the purposes of these lectures would be to overcome certain unfavorable impressions made by North Americans of questionable character who have operated in those countries in the past, and to convince the people with whom we propose to have business relations, that we are their friends and desire to render them faithful service. In addition to this, we should gain their confidence still further by making them actually co-partners with us in the development of their countries.

ALBERT M. BROSIUS, M. Am. Soc. C.E.
Civil and Sanitary Engineer

New York, N.Y.
February 20, 1932

Need for International Highway

TO THE EDITOR: I read with much interest Dr. Waddell's letter published in the November issue of CIVIL ENGINEERING. I believe that his suggestion for a "Latin-American Engineering-Contracting-Financing Syndicate" is the expression of a desire to help not only the engineering profession in the United States but also our neighbors to the south.

In considering Dr. Waddell's proposal, it must be remembered that he practiced his profession in Japan and China for many years. In fact, he only recently returned from the latter country where he acted as consultant to the Ministry of Railroads. Surely, over a period of many years, he could not have achieved the eminence that he has in these countries if his attitude had been patronizing.

In October 1930, I had the pleasure and satisfaction of attending the meeting of the last Permanent International Association of Road Congresses, which met in the city of Washington. This congress was attended by men from practically every civilized country in the world; and Central America and South America were represented by many delegates. Discussion of the construction of highways in the Central American countries was to a large extent overshadowed by discussion of the necessity for a highway which would permit uninter-

rupted travel between each state of Central America and North America. The same interest was also predominant among the South American delegates.

In considering the subject of highway construction in these countries, we must realize that motor vehicles do not require pavements such as are being built in many of the rich and highly developed portions of the United States. Pavements costing from forty to eighty thousand dollars a mile must wait until traffic conditions require them. Construction of the thousands of miles of such an international highway would constitute a clearing, excavating, and drainage job, with the use of such road surfacing materials as are most available.

Many modern pavements have been constructed in the United States at a cost which can be justified by no economic reasoning. We must not fall into this error at the beginning of a new project.

If a highway designed to carry modern traffic is to be built, connecting the Central American and the South American states with each other and with the United States and Canada, it must be done with the hearty cooperation, both financial and technical, of the Northern governments. Without such cooperation, it will not be possible to build an international highway within the next forty or fifty years.

There is no doubt in my mind that such a highway would be of tremendous value to the United States as well as to her Southern neighbors, and it is obvious that our engineers, who are much more experienced than the engineers of our neighboring countries, could adapt themselves to any legitimate program of major highway construction. Dr. Waddell has had a brilliant inspiration, and it is to be hoped that he will be able to bring it to materialization.

THERON M. RIPLEY, M. Am. Soc. C.E.
Consulting Engineer

Buffalo, N.Y.
February 27, 1932

Ideal for Developing Latin America

TO THE EDITOR: The proposal of Dr. Waddell, in the November issue, for a syndicate to develop Latin America is of peculiar interest to me, as are also the ensuing commentaries. As one who was born in Mexico and studied in its universities as well as in those of the United States, and has finally become permanently established here in the practice of civil engineering, I feel that I have an intimate knowledge of this question.

There is no doubt that there is yet a great deal to be done in Spanish America before these countries become thoroughly modernized. It is a fact, too, that if these countries are left to their own resources to develop themselves, it will take several hundred years before they are successful. Outside help is necessary.

In giving this help, American engineers cannot be selfish. The welfare of Latin America should be the primary object. For this purpose, as some of the contributors have suggested, it is essential to turn back into that country a generous share of the income and profit of American management. The opposite viewpoint, that of extracting profits without return to the local community, is unthinkable.

No country in the world has been developed when the money earned has not been spent in the same country. Unfortunately, the Latin-American countries have been the prey of many unscrupulous foreign enterprises whose only aim has been the exploitation of the country, taking back into their own countries every penny that has been made. Here lies the reason why foreign enterprises find so much difficulty with government regulations and are so much handicapped in the development of their projects. The viewpoint here suggested is not only common sense but justice as well.

DAVID FONSECA

Chicago, Ill.
March 6, 1932

Present Time Propitious for Latin-American Syndicate

TO THE EDITOR: In his letter, in the November issue, Dr. Waddell suggests for his proposed syndicate 15 engineers, who would presumably represent most of the major engineering fields.

The syndicate, when formed, would subscribe to a general fund sufficient to maintain a field force of 4 or 5 engineers, familiar with Latin America, general promotion procedure, economics, and financing. These engineers would serve as contact men in locating projects suitable for submission to the syndicate board, and the expenses of this group would be borne by those members of the syndicate directly interested. The cost would certainly not be more than is customarily incurred in scouting projects, and the members of the syndicate would have the obvious advantage of specialized advice.

As to opposition from the countries in which the work would be performed, twenty years' experience in the field convinces me that there would be no opposition as these countries have depended, and for years will continue to depend, on foreign capital for the development of engineering enterprises.

The question of why money should be invested in Latin America requires more explanation. We are told that South American securities have depreciated 70 per cent. The reply is that the market price is not a true index of their worth. There was a recent case in point when one issue of Buenos Aires bonds was quoted at 33, while the paper carrying this quotation also contained a notice by the fiscal agents of the redemption at par and accrued interest of the sinking fund allotment of those same bonds.

A further reason why these obligations will, in the main, be met is the fact that the Latin-American countries will need, for a long time to come, the investment of foreign capital; and in order to get this capital, they must keep their credit, or regain it, by meeting the outstanding obligations.

Since present conditions make it impossible for various governments to accept the ruinous discounts required if they are to proceed with certain projects, private capital has the opportunity to obtain charters, or concessions, to build and operate these works under advantageous terms. Several of these that would ordinarily be unobtainable have recently come into my hands, and no doubt other engineers with similar contacts have had the same experience.

Because Latin-America is a new land, so far as economic development is concerned, with everything still to be done, opportunities for the profitable employment of capital are more numerous, and competition is less severe, than in more developed lands.

A splendid opportunity exists. The problem is to prove, by engineering and economic argument, to those with money that the plan is feasible.

CLIFFORD C. MUHS, M. Am. Soc. C.E.
Muhs and Company

Chicago, Ill.
March 3, 1932

SOCIETY AFFAIRS

Official and Semi-Official

Engineers and Architects License Laws in New York State

By THOMAS C. DESMOND, M. AM. SOC. C.E.
SENATOR, 27TH DISTRICT, STATE OF NEW YORK

HAPPILY the long standing difficulties between engineers and architects in New York State, covering mutual interests in the building field, have been resolved. The necessary amendatory legislation was introduced in the 1932 legislature by Senator Thomas C. Desmond and Assemblyman Willis H. Sargent and subsequently passed. These bills, Senate Print No. 1867, amending the Engineers Licensing Law, and Senate Print No. 1866, amending the Architects Licensing Law, are expected to receive Governor Roosevelt's signature and thus to become law.

The results to date have been reached only after extensive conferences in which many interested parties participated. In general it may be said that the two acts set up the professions with equal but not identical prerogatives. Each may pursue the ordinary practice of its work without exclusion of the other. The points of similarity as well as of difference may be seen from a few comparisons. It is stated that:

> A person practices professional engineering, within the meaning and intent of this article, who holds himself out as able to perform, or who does perform, any professional service, such as consultation, investigation, evaluation, planning, design, or responsible supervision of construction or operation, in connection with any public or private utilities, structures, buildings, machines, equipment, processes, works, or projects, wherein the safeguarding of life, health, or property is concerned or involved, when such professional service requires the application of engineering principles and data.

On the other hand, the practice of architecture includes many similar and identical professional services, but involves in addition "esthetic" design; and finally, such professional service is defined as requiring "the application of the art and science of construction based upon the principles of mathematics, esthetics, and the physical sciences."

An engineer, and similarly an architect, is considered to be "a person who engages in the practice of professional engineering," as that has been defined. This law also permits the use of a suitable seal and requires that "all plans, specifications, plats, and reports prepared by or under the supervision of such licensed professional engineer.....shall be stamped with said seal when filed with public officials.

A Board of Examiners is provided as at present constituted; it is entitled to the services of the Attorney-General and has power to compel attendance of witnesses. The board is to require of applicants for license as professional engineer the following qualifications: age, 25 years; citizenship or declared intention; good moral character; high school training; and eight years practical experience in professional engineering of satisfactory grade and character.

Somewhat similar requirements are prerequisites to the granting of a license to practice architecture. There are important differences, however; the minimum age is made 25 years, the education includes high school and two years in a satisfactory institution, five years' practical experience subsequent to high school study, and finally written and oral examination.

In effect the laws are worded to permit an engineer to practice architecture and vice versa. In the engineers' law there is the statement that the statute "shall not be construed to affect or prevent the practice of architecture by an architect duly licensed under the laws of this state."

Similarly, the architects law "shall not be construed to affect or prevent the practice of engineering by a professional engineer duly licensed under the laws of this state nor to apply to any person licensed as a professional engineer in this state except that such person shall not use the designation architect or architecture unless licensed." After 1937, when certain provisions go into effect, the two professions will be even more similar before the law.

It is noteworthy that the definition of professional engineering includes work on "buildings." This improvement settles one of the critical points of controversy in past years.

Civil engineers cannot but take satisfaction in this satisfactory solution of a vexing problem. Because of the notoriety attending past efforts to accomplish the same results, the situation in New York State has been unusually prominent. It is possible that the favorable issue will therefore be an encouragement to similar negotiations throughout the country.

A number of men have rendered great service to both the engineering and architectural professions in assisting in the preparation and passage of these two bills. Especially helpful, however, were Dr. Harlan H. Horner, Director of Professional Education in the New York State Educational Department; D. B. Steinman, M. Am. Soc. C.E., Vice-Chairman of the New York State Board of Licensing for Engineers; Arnold G. Chapman, M. Am. Soc. C.E., Legislative Representative of the New York State Society of Professional Engineers; Messrs. Albert L. Brockway, President of the New York State Board of Examiners and Registration of Architects, and James F. Bly, representing the architects; Charles B. Heisler, Chief of the Bureau of Professional Registration in the New York State Educational Department; and the various members of the joint Engineer-Architect Conference Committee, headed by V. T. Boughton, Assoc. M. Am. Soc. C.E.

It has been a privilege and a pleasure to be associated with them in this worthwhile endeavor.

Yellowstone and the Annual Convention

At this time of the year, attention is usually drawn to the Spring Meeting, but as this meeting is to be omitted this year, interest is naturally transferred to the Annual Convention, the next gathering of the Society as a whole, which is to take place in Yellowstone National Park, July 6–9. Details of the meeting, the Sixty-Second Annual Convention of the Society, will shortly be available. Plans have already matured to such an extent that it is evident the program will be unusually attractive and will prove a strong inducement to members to be present—those from the East as well as those from the West.

In addition to being a rallying point for many members and their families living west of the Mississippi, it is expected that the meeting will draw many from the East also. They will welcome this double opportunity of renewing professional contacts and personal friendships at a Society gathering and at the same time enjoying the unparalleled scenic beauties of this most famous of our national parks.

There is still another reason why a good attendance may be anticipated. By fortunate coincidence, the Fortieth Annual Meeting of the Society for the Promotion of Engineering Education is also scheduled to be held in the West, at Corvallis, Ore., from June 29 to July 1. It is thought that many engineering teachers will find it practicable as well as economical to combine attendance at this earlier meeting with a visit to Yellowstone Park. The proximity of the two meetings, both in time and place, seems therefore most happy.

Horatio Allen Scholarship at Columbia University

It is the privilege of the Society to dispose of a scholarship in civil engineering at Columbia University for the year 1932–1933—an award having a monetary value of about $700. This scholarship has been established in honor of Horatio Allen, of the Class of 1823, Columbia University, the fifth President of the Society. During the past two years it has been held by David C. MacMurray, who is now completing his work. Although the original appointment is for one year, provision is made whereby the candidate may be reappointed, permitting him to complete three full years of scholastic work.

At Columbia, the course in engineering is based on a four-year

undergraduate curriculum leading to a bachelor's degree, together with a fifth, or professional year, for which the degree of Civil Engineer is awarded. Candidates must have completed in their own colleges the equivalent of the first two years of the undergraduate course; more mature students will get correspondingly greater credit. In general it is assumed that a student holding a bachelor's degree will require from one to one and one-half years to complete this graduate work.

Each applicant for this scholarship must submit a photograph together with his scholastic record and a letter of recommendation from the dean of his school. The record must be approved both by the Committee on Scholarships of the Society and by the Dean of the Engineering Schools, Columbia University.

Information may be received by addressing the Secretary of the Society, through whom applications will be transmitted to the appropriate committee. Applications will be received until June 1.

A National Project for Water Works Betterment

A public welfare plan for water works betterment, including adequate fire protection facilities, authorized by the American Water Works Association and worked out in detail by Harrison P. Eddy and Malcolm Pirnie, Members Am. Soc. C.E., is now being considered as a means of relieving unemployment. The plan, which would make possible normal efficiency in water works systems throughout the nation, has been approved in principle by the Board of Direction of the Society and is believed to be one of the projects that can be financed through the Reconstruction Finance Corporation. Stated simply, it is a "made work" project, nation-wide in scope, which would give employment to a large number of people and would contribute directly to the welfare of urban populations.

Details of the plan, as furnished by Mr. Pirnie, are here given for the information of members. To quote him: "It is one of those plans which may well be included in any organized endeavor to reduce unemployment through the initiation of betterments which are financially sound and which are of benefit to the country as a whole. While 60 per cent of the estimated costs of the water works betterments would be spent for labor in the communities installing the works, a large proportion of the balance of the costs would be spent for labor outside of those communities—labor in coal and iron mines, limestone quarries, sand pits, pig-iron industry, cast-iron pipe foundries, steel, and cement mills. Add to this the freight charges on raw materials and finished products, which would help support labor on the railroads through freight charges paid, and it is not unreasonable to estimate that 80 per cent of the prices paid for materials delivered to the communities would go for labor."

The immediate needs are considered to be:

$46,000,000 for deferred construction in privately-owned water works systems.

$500,000,000 for construction needed in all water works systems to put them on an efficiency basis.

$200,000,000 for additional water works, the need for which has been shown by the recent drought.

This list does not include the sums needed for such supply works as those contemplated by New York, Los Angeles, and Philadelphia, which alone would be greater than the last amount, and which might also be financed under this plan.

According to Mr. Pirnie: "Under present conditions, all that may be considered as deferred water works construction in privately-owned water works systems that might be initiated, if it could be financed, is given in the following tabulation:

Year	Normal Requirement	Estimated Expenditure	Estimated Deficiency
1931	$40,000,000	$20,000,000	$20,000,000
1932	40,000,000	14,000,000	26,000,000
Total	$80,000,000	$34,000,000	$46,000,000

"In this table, the $34,000,000 in 1931 and 1932 is almost entirely for extensions of distribution systems into paying territories, installation of meters, and some expenditures which materially reduce cost of operation. The $46,000,000 deficiency therefore probably represents deferred work on supply mains, reservoirs, purification works, and possibly some absolutely essential mains for reinforcement of distribution systems."

In order to make a conservative estimate of needed water works construction, there has been determined the tonnage of cast-iron pipe which is needed to reinforce the mains of distribution systems to provide adequate fire protection in the cities that have been investigated by the National Board of Fire Underwriters. It was found that in the country as a whole there is need for 1,600,000 tons of cast-iron pipe for reinforcing mains only. For the replacement of 4-in. and 6-in. pipes by 6-in. and 8-in. sizes in existing distribution systems, rightly termed "deferred maintenance," 800,000 tons more of cast iron are needed. In addition to this, some distribution systems should be extended to supply water service and fire protection in the outskirts of communities; 50,000 tons is a conservative estimate for this extension of service, bringing the total tonnage of cast-iron pipe needed at this time up to 2,900,000 tons.

Information received from certain water works holding companies indicates that expenditures for water works betterments in 1931 were one-half the normal and that those under 1932 budgets will be one-third of normal. In municipalities no new expenditures for water works betterment that can possibly be avoided will be authorized in 1932. This will have the effect of deferring maintenance work in addition to postponing needed betterment programs.

The estimated cost of installation of the 1,600,000 tons of reinforcement mains and appurtenances needed to produce better fire protection is $208,000,000, which is about 8 per cent of the cost of existing distribution systems, or 4.5 per cent of the cost of all water works plants. It is obvious that private companies and municipalities cannot increase their capital outlay 4.5 per cent for betterment that will not add to gross revenue at a time when water sales, depended upon to pay a large percentage of fire protection costs, are materially below normal. It is generally recognized that there is a tremendous need for better fire protection, which could be secured by a large expenditure for reinforcement mains, but there is no method of financing this outlay. At present, fire-protection charges in nearly every city are below cost, and in a large number of cities there is no charge at all for this service.

Here, then, it would appear, is a worthy use for Federal-aid funds in a public welfare project that would yield nation-wide benefits. It is thought that this plan may be put in operation by the Reconstruction Finance Corporation Board to demonstrate that definite benefits have resulted from the Board's activities.

March Society Meeting

As required by the Constitution, the regular Society Meeting for March was called, on March 24, at the Engineering Societies Building in New York. Vice-President Tuttle presided. After announcements by the Secretary, there being no unfinished or new business to come before the Society, the meeting was adjourned and was immediately followed by the regular meeting of the Metropolitan Section.

Freeman Traveling Scholarship Available

Announcement is made of the availability of the Freeman Traveling Scholarship for one year, beginning July 1, 1932. The present holder of this award is Hans Kramer, Assoc. M. Am. Soc. C.E., First Lieutenant, Corps of Engineers, who has held it for the past two years. With his return to the United States shortly, the position becomes open for a new appointee. The requirements and advantages of this work have been widely and favorably known since its inception several years ago. It presents to some young American engineer an admirable opportunity to gain an intimate understanding of European practice in hydraulics under the most unusual and favorable circumstances.

The requirements are correspondingly high. Briefly they may be summed up as follows: American citizenship; age from 24 to 35 years; graduation from a recognized technical school; and present employment as a junior professor, instructor, or assistant in a standard college in which hydraulics has an important place in the curriculum. Membership in some grade in the Society is considered desirable. Aside from these minimum requirements, it is desirable that the applicant should have good training in mathematics, especially as regards hydraulic problems, and in addition, experience in hydraulic design and construction. A

working knowledge of the German language is also considered essential.

Financially, this award yields $1,600 during the year, of which $500 is paid on July 1 and the remainder in 11 monthly instalments of $100 each. The winner should be prepared to bear any other slight expenses incident to tuition, travel, and living in Europe during the year.

In making applications to the Secretary of the Society, a photograph must be submitted with data as to age, place of birth, education, experience, and references. The applicant must supply six letters of reference, including one from the administrative officer under whom he teaches. In addition, a personal interview may be required.

Complete information and requirements as to the Freeman Traveling Scholarship are available on application to the Secretary's office. This brief résumé is given here to enable those who are interested to make further inquiry of the Secretary of the Society. The advantage of the early receipt of applications, making possible an early decision by the committee, is obvious from the standpoint of both the Society and the candidate.

The Washington Award for 1932

Announcement has been made that the Washington Award Commission has given the 1932 award to William David Coolidge, D.Sc., Associate Director of the General Electric Research Laboratory, at Schenectady, N.Y., for his scientific spirit and achievement in developing ductile tungsten and the modern X-ray tube. On February 24 the presentation was made at a dinner in Chicago, at which over two hundred members of the Western Society of Engineers and the four Founder Societies were present as guests. The presentation was made by Frank D. Chase, president of the Western Society of Engineers; and Elmer T. Howson, M. Am. Soc. C.E., acted as chairman of the Washington Award Commission.

The principal address of the evening was given by Dr. Harry Woodburn Chase, President of the University of Illinois, who spoke on research as it affects present-day life. He traced briefly the lives of a number of our early scientists and pictured their difficulties in fundamental or industrial research as compared with those of modern researchers.

This is the eleventh time the Washington Award has been given. It is bestowed annually by the Western Society of Engineers on recommendation of a committee of representatives of that society and of each of the four Founder Societies. In the following list of those who have previously received this award are many members of the Society: Herbert Hoover, Robert Wooltson Hunt, Arthur Newell Talbot, J. Waldo Smith, John Watson Alvord, Orville Wright, Michael Idvorsky Pupin, Bion Joseph Arnold, Mortimer Elwyn Cooley, and Ralph Modjeski. At the ceremony of presentation the Society was presented by A. J. Hammond, M. Am. Soc. C.E.

Report of United Engineering Trustees for 1931

An annual report has recently been issued by the United Engineering Trustees, Inc., which is the holding company for the four Founder Societies, with respect to the property used as headquarters. Under the control of this body the Engineering Societies Building is administered on a cooperative basis and not for profit. Offices not used by the Founder Societies and their joint organizations are allotted to associated societies. Meeting halls, when not in use by the Founder Societies, are rented and the revenue thus secured helps reduce the burden of maintenance, operation, and fixed charges.

In addition, the Board of Trustees administers trust funds for the Engineering Foundation and the Library, and handles other common property. In these financial operations, the Trustees have the services of trust companies, legal counsel, certified public accountants, and a consulting architect. During the past year, the Trustees acted as treasurer for the Professional Engineers Committee on Unemployment, a committee of the metropolitan sections of the four Founder Societies.

According to the report for 1931, the Engineering Societies Library and the real estate and funds entrusted to the holding corporation are all in excellent condition. All departments closed the year without deficits. Assets for which the corporation is responsible (real estate at cost, funds at book value, and the library as appraised) total $3,908,957.52. Membership in the four Founder Societies totals 62,200 and that of the associated societies, 24,500; so that 86,700 engineers have headquarters in the Engineering Societies Building.

The accountant's report and the yearly balance sheet reveal the following activities during 1931:

OPERATION OF BUILDING		
Credit balance January 1, 1931	$ 10,636.73	
Less adjustment 1930	−25.00	
		$10,611.73
Building revenue 1931	126,837.37	
Building expenditures 1931	115,546.96	
		$11,290.41
Total		$21,902.14
Annual payment to depreciation and renewal fund	$ 12,000.00	
Payment to reserve for future fire insurance	1,550.00	
		13,550.00
Credit balance December 31, 1931		$8,352.14
OPERATION OF LIBRARY		
Maintenance revenue	$ 50,674.96	
Maintenance expenditures	52,011.70	
Debit balance for year 1931	−1,336.74	
Credit balance from preceding years	5,688.95	
Total credit balance December 31, 1931		$4,352.21
Service Bureau revenue	$ 17,031.56	
Service Bureau expenditures and adjustments	17,422.17	
Debit balance for year 1931	−390.61	
Credit balance from preceding years	4,733.96	
Total credit balance December 31, 1931		$4,343.35
FUNDS AND PROPERTY		
Funds held by United Engineering Trustees, Inc., December 31, 1931 (book value)		
Depreciation and renewal fund		$303,397.14
General reserve fund		3,934.40
Engineering Foundation Fund		529,283.65
Engineering Foundation Fund (4th Swasey Gift)		253,696.50
Henry R. Towne Engineering Fund		50,115.63
Library Endowment Fund		174,544.32
Edward Dean Adams Fund		100,000.00
Total		$1,414,971.64
Real estste owned, cost to December 31, 1931		$1,987,793.92
Operating cash and petty cash		10,092.63
Accounts receivable		5,585.83
Endowment Committee loan receivable		1,700.00
Unexpired fire insurance premiums		1,213.50
Fire insurance fund		4,600.00
Total property which United Engineering Trustees, Inc., owns or holds (exclusive of Library)		$3,425,957.52
Value of Library as (appraised for insurance)		$ 483,000.00
John Fritz Medal Fund (custodian)		3,500.00

Appointments of Society Representatives

ARTHUR S. TUTTLE, M. Am. Soc. C.E., was appointed to represent the Society at a celebration of the bicentenary of the birth of Washington and the dedication of Houdon's bust of Lafayette on February 22, at New York University.

ALONZO J. HAMMOND, M. Am. Soc. C.E., was appointed to represent the Society at the presentation of the Washington Award to Dr. William David Coolidge in Chicago, on February 24.

O. H. AMMAN, M. Am. Soc. C.E., has accepted a Society appointment as second alternate on the American Marine Standards Committee.

R. K. TIFFANY, M. Am. Soc. C.E., has accepted an appointment as Society representative on the Sectional Committee of the American Standards Association on the Rating of Rivers.

A Preview of Proceedings

A description of a new concept in the analysis of gravity-arch dams is an outstanding feature of the April issue of PROCEEDINGS. *The other paper to appear in this issue, that by C. C. Cragin, M. Am. Soc. C.E., entitled "Hydro-electric Power Development as an Aid to Irrigation," was "pre-viewed" in* CIVIL ENGINEERING *for March. Because of unforeseen circumstances, this paper could not be published in the March issue of* PROCEEDINGS, *as announced, but will definitely appear in the April issue. The rest of the issue will be given over to discussions, which will occupy an important place and will take up a large percentage of the available space.*

THE COMPENSATED ARCH DAM

A new concept is applied to the theory of arch-dam design in the paper on "The Compensated Arch Dam," by A. V. Karpov, M. Am. Soc. C.E., Designing Engineer of the Aluminum Company of America. Fundamentally, the methods used in deriving the theory involved the division of the gravity-arch dam into vertical elements, or cantilevers, and horizontal arch slices, as in the trial-load method first outlined in TRANSACTIONS by Messrs. Howell and Jaquith. Mr. Karpov claims to have demonstrated the method of designing gravity arch dams in such a way that the relationship between the profile of the dam site and the shape of the dam itself can be properly appraised. The features brought out in the paper may be tabulated as follows:

1. The dam is divided into vertical cantilevers by means of radial planes, and into horizontal arches by means of horizontal planes.
2. Hydrostatic pressures are represented by radial forces acting at the upstream face of the dam.
3. The forces acting on the dam are divided so as to exert varying loads on the vertical and horizontal elements.
4. The horizontal arches are analyzed as independent units that are not influenced by gravity.
5. The horizontal arches are shaped in such a way as to relieve them from bending moments due to external loads. The arch is then compensated so as to relieve it from moments due to rib shortening, displacements of the abutments, and differences in temperature.

CALDERWOOD DAM
An Arched Overflow Structure on the Little Tennessee River

6. The relationship between the radial, tangential, and twisting displacements of the loaded arch is determined for any assumed deflection of the abutments.
7. The cantilevers or vertical elements are next analyzed, assuming them as independent, unrestrained from adjoining elements, and resting on the foundation with sufficient friction between the horizontal joints to prevent slipping at any elevation. The radial, tangential, and tortional displacements of the vertical elements due to the part of the water load taken care of by those elements are analyzed.
8. The next step is to appraise the conditions under which the independent, compensated, horizontal arches can be transformed into elements that are restrained in a horizontal as well as in a vertical direction, but still compensated and relieved from any bending moments. Under this assumption the horizontal arches of the vertical elements have equal radial, tangential, and twisting deflections.
9. The theory developed in this paper is applied specifically to symmetrical profiles, but an extension of the method is given, indicating its application to unsymmetrical profiles.
10. This partly analytical and partly graphical method of design is developed for a compensated arch dam. It takes into consideration the shape of the gorge at the site. By controlling the design so that the vertical joints of the loaded dam are under uniform compression, construction conditions are also taken into consideration.

The most outstanding innovation introduced in this paper is the method of compensating the arch for the effect of rib shortening, changes in temperature, and for displacement of the abutments. This consists in providing vertical notches at stated intervals so that the horizontal sections are composed of a series of voussoirs separated by properly defined joints. The notches, or V-shaped joints, are computed in accordance with special formulas derived by Mr. Karpov. It is expected that members will find much of technical interest in this paper to study and discuss.

DISCUSSIONS ON MANY SUBJECTS

The large proportion of discussion in the April issue of PROCEEDINGS deserves particular comment. It is possible that as many as fifty contributors may be represented in the discussion of about 19 of the 32 papers now before the Society for critical analysis.

A. R. Losh, whose paper on practical considerations of highway location was published in November 1930, presents his closing comments. Other papers discussed are those on the Cross method of distributing fixed-end moment, on rational run-off formulas, financing of street and highway improvements, design of large pipe lines, soil mechanics, Federal reclamation, water pressures on dams during earthquakes, bending stresses in wire ropes, tunnel construction, design of wind bracing, and public supervision of dams. This list indicates the wide appeal that this most important department of PROCEEDINGS will have.

All Proceedings Papers in Transactions

Frequent inquiries received at Headquarters disclose the fact that many members do not realize the basis on which papers are printed in TRANSACTIONS. All technical material, including discussions, which appears in PROCEEDINGS, is later assembled and collated to be printed in combined form in the first available volume of TRANSACTIONS.

This procedure has been in effect since the inauguration of CIVIL ENGINEERING in October 1930. Prior to that time, only selected papers from PROCEEDINGS were chosen for inclusion in TRANSACTIONS, with the thought that a much needed economy of Society funds would obtain by omitting from TRANSACTIONS such papers as were not deemed of the same high and outstanding character as those included under the available budget. A material number of papers formerly were omitted from TRANSACTIONS under this plan.

With the inception of CIVIL ENGINEERING, however, it became possible to institute a higher standard of acceptance for PROCEEDINGS papers. The selection is therefore now made before the paper goes into PROCEEDINGS and no further eliminations necessary.

This rule does not apply to certain material such as bibliographies, indexes, progress reports, and other similar matters, to which a second printing in TRANSACTIONS would add no appreciable information of value. In the aggregate, this material is relatively small as compared with the total size of TRANSACTIONS. With such minor exceptions understood, the present ruling of the Publications Committee therefore is that papers in PROCEEDINGS automatically are included in TRANSACTIONS.

News of Local Sections

Central Ohio Section

An interesting talk on "Some Public Utility Problems," was given by John W. Bricker, of the Public Utilities Commission, at the December 17 meeting of this Section.

The estimated attendance at the January meeting was 3,000. This was a joint meeting with the American Institute of Electrical Engineers, American Society of Mechanical Engineers, Engineers Club of Columbus, and the Electric League of Columbus. A talk and demonstration on "Recent Telephone Research and Its Application," was given by Dr. Sergius P. Grace, Assistant Vice-President, Bell Telephone Laboratories, Inc. The demonstration was a most unusual and interesting one.

The February meeting was a joint session with the American Institute of Electrical Engineers, American Society of Mechanical Engineers, Engineers Club of Columbus, and the American Chemical Society. In celebration of the two-hundredth anniversary of the birth of George Washington, Carrington T. Marshall, Chief Justice of the Supreme Court of the State of Ohio, gave an excellent address on "George Washington, Engineer, Business Man, and Statesman."

Chattanooga Section

The charter celebration of the Chattanooga Section was held at Cavern Castles on February 18. The economic responsibility of engineers was discussed by Prof. Nathan W. Dougherty, instructor in civil engineering at the University of Tennessee. Following this, John N. Chester, of Pittsburgh, Vice-President of the Society, gave a very interesting talk on the history of the Society. Mr. Chester was introduced by W. H. Sears, president of the Section, who outlined the aims of the Chattanooga Section. A response to Mr. Chester's address was made by A. F. Porzelius, after which Commissioner Zach Taylor addressed the engineers on behalf of the city. The programs for the banquet were cleverly prepared in blueprint form as "plans and specifications." Musical entertainment was provided by Mrs. Beverly Eaves accompanied by Mrs. W. H. Wilson, and the Rev. A. G. Frost gave selections on an electrical instrument, the theremin. The occasion was thoroughly interesting and enjoyable.

Cincinnati Section

There were 70 in attendance at the February meeting of the Section. Since he was unable to be present, H. M. Waite, Vice-President of the Society, sent his paper which was read by William V. Schmeidke. This paper outlined the proceedings of the meeting of the Board of Direction held at the time of the Annual Meeting in New York in January. An unusually interesting and instructive talk on foundation piling was then given by George P. Stowitts; and following this, interesting discussions were contributed by H. D. Loring, on piling in general, and by Lester Bachman and John Raffety on foundation piling used for the Times-Star Building and the Eighth Street Viaduct, respectively. Committee reports were made by J. S. Raffety on the subject of Junior Members and by F. L. Raschig on unemployment.

Cleveland Section

Business procedure occupied a large part of the session of the luncheon meeting of the Cleveland Section, held on February 9.

On March 1, another luncheon meeting was held, with 58 members and guests in attendance. In the absence of George B. Gascoigne, Frank C. Tolles presented Mr. Gascoigne's interesting paper on "Cleveland's Easterly Sewage Treatment Works."

Dayton Section

After luncheon, at the regular monthly meeting of the Dayton Section held on January 18, H. E. Senf, Field Engineer of the American Rolling Mills, of Middletown, Ohio, showed several reels of moving pictures on the continuous rolling process, recently installed in the Middletown plant.

The February luncheon meeting was followed by an instructive and interesting inspection trip through the new Eaker Street Steam Station of the Dayton Power and Light Company.

Georgia Section

The regular February meeting of the Georgia Section was held at the Atlanta Athletic Club on Monday, February 1. The guests in attendance were: Fred C. Hermann, Director from San Francisco; R. I. Lowndes of Atlanta; and I. A. Daniels, Federal Engineer in charge of the Atlanta Post Office project. Following the business session, Frank Neely, General Manager of Rich's, Inc., spoke on unemployment relief.

Illinois Section

The guest of honor at the luncheon meeting of this Section, on February 26, was Professor Enger, the new Director of the 8th District. After a brief talk by Professor Enger, Chester L. Post, president of the Section, discussed the subject of unemployment relief.

Iowa Section

A well attended meeting of the Iowa Section was held on the evening of February 9. "Control Surveys for Iowa Engineers" was the subject of an excellent address by R. S. Patton, Director, U.S. Coast and Geodetic Survey, Washington, D.C. The address was enthusiastically received.

Ithaca Section

The Ithaca Section was formally organized at Owego, N.Y., on Wednesday evening, January 10. The following members were elected officers for the coming year: Earle N. Burrows, President; Henry W. Preston, First Vice-President; Edward H. Prentice, Second Vice-President; and John E. Perry, Secretary-Treasurer

Los Angeles Section

The subject of the meeting of the Los Angeles Section, held February 10, was "The State Water Plan." A brief paper on "Water Spreading in San Fernando Valley," was presented by D. A. Lane, engineer of the Los Angeles Water Department. Then A. L. Sonderegger, president of the Section, gave a brief history of early attempts to formulate or develop some state water plan, after which an interesting speech on the fundamental laws and the development of the present laws governing water rights was given by Senator Ralph H. Clock, member of the legislative water committee. A résumé of the activities of the State Department of Water in collecting data and information from the inception of the state plan to the present date, was then given by Edward Hyatt, State Engineer.

A number of distinguished guests were present at the meeting of the Section held on March 9. Among these were Col. Carlos Huntington, Director of the Department of Professional and Vocational Standards of the State of California, who spoke on the registration of engineers; and Robert H. Bacon, of the Bridge Division of the Bureau of Engineering of the City of Los Angeles, who described the construction of the Figueroa Street Tunnel. The feature of the remainder of the meeting was discussion of a new water plan for the state. F. E. Trask, chairman of the special committee of the Section on the subject, read the report of his committee which emphasized the need of southern California for a new water supply.

Metropolitan Section

Postponed from its usual date, the regular meeting of the Metropolitan Section was held on Thursday, March 24. The principal address was by Thomas Adams, formerly General Director of, and now Consultant to, the Regional Plan of New York. Under the topic, "Limiting the Size of Business Buildings in New York," he dealt with the regulating effect of transportation facilities and the reasonable scope of skyscrapers. This paper called forth an interesting discussion led by Harvey W. Corbett, well known architect, and by Edward M. Bassett, Counsel, Zoning Committee, both of New York City. Preceding the Section meeting was a brief regular business meeting of the Society. As usual, refreshments concluded the evening.

Nashville Section

The Nashville Section elected A. J. Dyer, President, and W. D. Glenn, Secretary-Treasurer, for the coming year.

Philadelphia Section

A report from the Philadelphia Section indicates that a most enjoyable annual dinner and meeting was held on February 20.

The entertainment, including music, toe dancing, and demonstrations of the plasticity of clay, was followed by dancing and cards.

SAN FRANCISCO SECTION

The regular meeting of the San Francisco Section was held on Tuesday evening, December 15. After some excellent musical numbers by the Arion Trio, the business session was called to order. Following this, the first speaker of the evening, C. H. Purcell, Chief Engineer of the San Francisco-Oakland Bay Bridge, addressed the meeting on "Preliminary Foundation Studies and Main Span Design of San Francisco-Oakland Bay Bridge." "Mud, Tides, and Submarine Topography" was the subject of the next talk by R. E. Hambrook, General Plant Manager, Pacific Telephone and Telegraph Company. There were 156 members and guests present for dinner, and 80 at the business and technical meeting which followed.

SEATTLE SECTION

At the regular monthly meeting of the Seattle Section, W. F. Way gave an interesting account of his trip en route to the Annual Meeting of the Society in New York, and Maj. Joseph Jacobs told of the action taken by the Board of Direction at the Meeting. After this, a very interesting description of the methods of operation of the Port of Seattle, was given by J. R. West, Chief Engineer of the port. Mr. West described the methods of construction of the various units, and with the aid of charts gave a very clear picture of exports and imports through the port. There were 35 in attendance.

SPOKANE SECTION

The speaker of the day at the regular noon meeting of the Spokane Section, which met on February 11, was V. H. Greisser, president of the Section. His subject was the Columbia Basin Project. Figures and facts were presented on the subject of the dam, laterals, canals, the cost per acre, the total cost, and the time that will elapse before the project can be brought to a state of production where it will pay the interest on the invested capital.

TACOMA SECTION

An illustrated address on the strength of Douglas fir was given at the February 8 meeting of this Section by Ernest Dolge, of Tacoma, one of the most prominent lumber manufacturers on the Pacific Coast. He showed the results of tests made at the Massachusetts Institute of Technology, on structural grades of fir and told of his efforts to establish proper unit stress for the various grades of structural timber. Discussion was contributed by C. S. McCormick, O. R. Elwell, J. Hart, and other bridge and tunnel engineers.

UTAH SECTION

The Utah Section has elected Richard B. Ketchum, President, and Frederick H. Richardson, Secretary-Treasurer, for the coming year.

Student Chapters Active During 1931

Of such interest is the report of the Committee on Student Chapters concerning activities during the year 1931 that it has been reproduced in quantity for distribution upon request. The list of Contact Members appointed, the tabulation of statistical data, the report on the use of lantern lectures, the account of two Student Chapter conferences, and recommendations for student activities in the coming year, are high points in this report. In addition, detailed tabulations of Student Chapter statistics are given in full.

The major objective of the 1931 committee was to complete, as far as possible, the appointment of Contact Members for the various Student Chapters. These are members of the Society who are engaged in active practice near the Student Chapters, and who are interested in helping these young men to orient themselves upon entering the profession. Their efforts are to supplement but not to supplant the normal work of the Faculty Sponsors in this regard. In one year the committee secured the appointment of 80 out of a possible 99 Contact Members.

The second objective was to secure and analyze certain statistics from the Chapters, particularly in regard to types of meetings held and attendance at them, and also to list Chapter membership by classes. In addition, opportunity was given for the making of suggestions relating to the improvement of meetings and cooperation from pertinent units in the Society.

Detailed reports were received from 94 Chapters, a remarkably high percentage. The total number of meetings reported was 1,100, with an attendance of 39,691. Nearly one-third of these meetings were addressed by student speakers with an average attendance of 23, while one-fourth featured an outside speaker and drew an average of 47 listeners. Lantern lectures were popular, an average of 32 persons being present at 118 showings of these convenient talks. It is interesting to note that during the calendar year 1931, lantern lectures were shown at 185 different meetings.

Membership in the Chapters totaled nearly 4,700, with upperclassmen naturally predominating. An analysis of Chapter membership shows that it consists of 38 per cent seniors and 32 per cent juniors, and that there is an increasing tendency to attract sophomores and freshmen.

The two Student Chapter conferences, at Norfolk in April and at St. Paul in October, were characterized by excellent attendance and vigorous discussion. As a result of observation made at these conferences, it was strongly recommended by President Stuart and the committee that a serious attempt be made within the Chapters to stress speaking ability and a proper public presence. The representatives to the Student conferences gave splendid examples of how this should be done.

Copies of the report are being sent to the president, the Faculty Sponsor, and the Contact Member of each Student Chapter. A supply is available at Society Headquarters so that any one interested may receive a copy.

Student Chapter News

NEWARK COLLEGE OF ENGINEERING STUDENT CHAPTER

A number of interesting meetings have been held during the past winter by the Newark College of Engineering Student Chapter. At the first meeting, in October, Jacob L. Lenox, of the staff of Alexander Potter, presented an illustrated lecture on "The Fourth Ward Sewage System of West Orange, N.J." A week later the Chapter attended a lecture sponsored by the Newark Women's Junior League, at which Col. Hugh L. Cooper, Hydraulic Engineer, spoke on "Russia."

"The Construction of the George Washington Memorial Bridge," was the subject of an illustrated talk by C. W. Dunham, of the Port of New York Authority, at the November meeting. A fortnight later, the Chapter inspected the Plainfield Joint Sewage Disposal Plant, at Plainfield, N.J.

In December, A. S. Bank, of the Water Department of the City of Newark, spoke on "Newark's Water Supply" and showed slides of portions of the watershed and distribution system.

The speaker at the January meeting was Morris M. Sherrerd, consulting engineer for the New Jersey State Water Policy Commission, who spoke on the work of this commission.

At the last meeting in March, Lt. Richard Aldworth, Manager of the Newark Airport, gave an interesting address on the subject of "Airport Location and Management."

NEWARK COLLEGE OF ENGINEERING STUDENT CHAPTER

American Engineering Council

National representative of 26 engineering societies, with a constituent membership of 60,000 professional engineers, reports civil engineering news of the Federal Government

CONTRACTING FOR ARCHITECTURAL AND ENGINEERING SERVICES

The Committee on Public Buildings and Grounds of the House of Representatives, of which F. G. Lanham is chairman, has completed hearings on the various bills which propose to direct the Secretary of the Treasury to contract for architectural and engineering services in the designing and planning of public buildings. H.R. 6,187, introduced by Congressman Green of Florida, is the bill which received the endorsement in principle of the American Engineering Council.

MUSCLE SHOALS

S.J. Resolution 15, relative to Muscle Shoals, sponsored by Senator Norris, was reported in the Senate, March 9, by the Agriculture and Forestry Committee. This measure has been consistently opposed by American Engineering Council.

PUBLIC WORKS AID TO UNEMPLOYMENT

The LaFollette bill, S. 2,419, which would create an Administration of Public Works and authorize a several-billion-dollar bond issue with which to finance its activities, has been receiving attention through hearings before the Senate Committee on Labor and Education. From March 8 to 11, a number of engineers including John P. Hogan, Sanford E. Thompson, Malcolm Pirnie, and R. C. Marshall, Jr., Members Am. Soc. C.E., appeared before the committee and testified to the effect that the construction industry was likely during 1932 to reach a new low level of activity and that under present financial conditions it is practically impossible even for cities in sound financial position to secure money for the construction of needed public works.

RECLAMATION MORATORIUM

The original bill to authorize the Secretary of the Interior to grant a moratorium on payment of construction charges on reclamation projects, which was opposed by American Engineering Council, has been rewritten and now appears in Congress as H.R. 9,489. This measure was reported out by the Committee on Irrigation and Reclamation, February 19, 1932. A similar bill, S. 3,422, is on the Senate calendar. It seems inevitable that Congress will pass some legislation on this subject in this session.

DECREASED APPROPRIATIONS FOR TOPOGRAPHIC MAPPING AND STREAM GAGING

In an effort to decrease Government expenditures, the House of Representatives recently passed the Interior Department Appropriation Bill, H.R. 8,397. The appropriations for this department, which contains bureaus and services of vital concern to the engineering profession, were reduced from over $69,000,000 for 1932, to slightly over $50,000,000 for 1933. Some of the most serious reductions were in items for topographic surveys, geologic surveys, stream gaging, and printing of maps. The inconsistency of such a drastic cut is shown by the fact that within two weeks after the House action on this bill, it passed a measure authorizing $132,000,000 for emergency road work.

The American Engineering Council protested vigorously against the reductions for stream gaging and topographic mapping. The Senate Committee on Interior Department Appropriations has reported out H.R. 8,397 and has increased the items for topographic mapping by $100,000, making the apparent amount for next year that recommended by the budget. However, the bill carries over a supposed unexpended balance of $150,000, but there is not this much left unexpended at this time for state cooperative mapping, and it is very likely that this sum will be materially reduced by June 30, 1932. This action is very disappointing, especially to civil engineers, who recommended that the amount of money to be expended for topographic maps be increased to several million dollars as an unemployment relief measure.

If the increases as passed by the Senate Committee are finally retained in the bill, the U.S. Geological Survey will be able to match only the state cooperative work for 1933 that is now requested, but no more. If the increases proposed by the Senate Committee are not sustained, the topographic mapping program will be seriously impaired.

NECESSITY FOR STREAM GAGING EMPHASIZED

In the hearing before the Senate Committee, it was emphasized that if the stream-gage readings were not obtained during the next fiscal year that particular record would be gone forever. This brought results. The stream-gaging item was increased by the committee to $700,000 which is only $20,000 under the 1932 appropriations—a regrettable reduction but not a crippling one.

The Senate Committee has also restored $48,240 for printing, binding, illustrating, and engraving maps. This, too, is of vital importance. Had appropriations of the same magnitude as those for 1932 been made, the arrearage in map printing could have been overcome in another year or so. Some maps will now have to be further delayed. It is highly desirable that engineers write promptly to their respective Senators and Congressmen that the appropriations provided by the Senate Committee for topographic surveys, printing of maps, and stream gaging, should be supported.

FEDERAL GOVERNMENT REORGANIZATION

Both major political parties have endorsed the principle of Federal Government reorganization, and President Hoover has urged the realization of these pledges in each of his two last annual messages. On February 17 the President sent a special message to Congress in which he pointed out the obvious need for reorganization and called attention to the difficulties that would be encountered under any proposed plan. After setting forth the general principles that the reorganization plan should follow, the President recommended the creation of an Administration of Public Works, and stated concerning this agency:

> The public works administration should be partially a service agency to the other departments of the Government, executing certain construction work, the subsequent operation of which should be carried on by those departments. It should also be partially an agency administering certain contract work which cannot be delegated to any one department. Naval, military, and some other highly specialized construction work should not be transferred to this agency. The supervision and construction of rivers and harbors work should be continued under direction of the Army engineers, who should be delegated by the Secretary of War to the service of the Administrator of Public Works for this purpose.

Congress has been unusually active with this subject and to date there have been introduced more than 24 different bills, but action has been obtained on only three of them. Senate Joint Resolution 76, by Senator George, authorizing the President to reorganize the executive agencies of the Government, has been reported out unanimously by the Senate Committee on Finance and is now on the Senate Calendar.

ECONOMY COMMITTEE SET UP

The House of Representatives passed House Resolution 151, setting up an Economy Committee to investigate and determine which, if any, of the Federal bureaus and commissions now functioning, might be abolished, restricted, or consolidated, without impairment of orderly action, and to make recommendations as to the possible economies to be effected thereby. The Economy Committee appointed under authorization of this resolution is composed as follows:

Congressman Byrns, Democrat, of Nashville, Tenn.
Congressman Cochran, Democrat, of St. Louis, Mo.
Congressman McDuffie, Democrat, of Monroeville, Ala.
Congressman Douglas, Democrat, of Phoenix, Ariz.
Congressman Wood, Republican, of LaFayette, Ind.
Congressman Williamson, Republican, of Rapid City, S.D.
Congressman Ramseyer, Republican, of Bloomfield, Iowa

Hearings have been held before the House Committee on Expenditures in the Executive Departments on H.R. 6,670, a bill creating an Administration of Public Works. This bill was endorsed in principle by the American Engineering Council although the Council would like to have the bill amended in a number of places. Representatives from the Departments of War, Agriculture, and the Interior have been heard by the committee on this bill.

ITEMS OF INTEREST

Engineering Events in Brief

Civil Engineering for May

BECAUSE of the growth of Panama Canal traffic, the storage in Gatun Lake will soon be too small to supply the water needed for lockage. A new storage reservoir will be formed on the Chagres River, above Gatun Lake, by the construction of the Madden Dam, in a formation of sandstone and fissured limestone. In the May issue of CIVIL ENGINEERING will appear an article by E. S. Randolph, M. Am. Soc. C.E., in which he will give the reasons for the selection of an overflow gravity section for the dam, and explain to what extent the reservoir will improve operating conditions in the Canal.

Another article which it is planned to include in the May issue, is by A. P. Welty, M. Am. Soc. C.E. He advocates the subdivision of irrigated land in accordance with its topography. The ordinary, rectangular method of subdivision of land has many disadvantages, which are aggravated when the land is irrigated from open ditches by gravity flow. To support his contention, Mr. Welty gives complete comparative estimates of the number of irrigation structures, the amount of canal excavation, and the total cost by each plan for a 5,000-acre portion of the Medina Irrigation Project in Texas.

The City of New Orleans is from 8 to 16 ft. below high water in the Mississippi River, and at least a third of its area is below sea level. It is entirely surrounded by levees and has, of course, no natural drainage outlet. Every drop of water that falls as rain or is piped into this sunken area for domestic purposes, must either evaporate or be pumped out over the surrounding levees. An article to appear in May, by George G. Earl, M. Am. Soc. C.E., points out that an expenditure of nearly $60,000,000 since 1900 for drainage pumps, sanitary sewer systems, and a pure water supply has so improved the unsanitary conditions formerly existing, that deaths from yellow fever, typhoid, and malaria, now rarely occur.

On the headwaters of the Mississippi River in Minnesota a reservoir system, unique in purpose and capacity, was authorized by Congress in 1878 and was later constructed by the Government. The system is operated by the War Department to regulate the volume of water in the river and improve navigation. An article under the joint authorship of Hibbert Hill, Assoc. M. Am. Soc. C.E., and Ralph P. Johnson, Jun. Am. Soc. C.E., describes the use to which this two-million acre-ft. of storage has been put.

Numerous other articles of interest to engineers are available and in preparation for publication. In addition to those specifically mentioned here, the May issue will contain certain of these, together with items regarding the Society's many activities, and brief discussions by readers.

Recent Graduates Favor Engineering

DURING the spring of 1931 the Education Research Committee of the Engineering Foundation requested a group of letters from young engineering graduates working for the General Electric Company with the idea of procuring from them some general statements on their reactions to their employment. It was thought that these letters might furnish useful material for presentation to prospective students of engineering and that the data would have particular value for the purpose because of the relatively recent student status of those giving the information.

In order to obtain comparable statements the following questions, among others, were suggested as a basis for the answers requested:

1. (*a*) Is the work what you thought it would be before you commenced it? (*b*) Do you like the work? (*c*) Why? (*d*) Would you like to continue it indefinitely? (*e*) Does there appear to be a chance for advancement?
2. (*a*) Knowing what you now know about engineering would you recommend it to a young man entering college, assuming of course that he was interested in it?
   (*b*) Upon what subjects in the engineering curriculum would you place the greatest stress?

Letters were submitted by 28 men, whose periods of employment since graduation ranged from six months to five years. The men probably constitute a picked group in the sense that they are all young men who have shown measurable success considering their time of service. They represent some twenty institutions well distributed over the country, and from that standpoint offer a good sampling of young graduates.

These letters are rather interesting reading, more because of the unanimity of their writers' reactions, perhaps, than from any inherently valuable contributions that they make towards solving the problem of how best to paint the engineering picture for the guidance of prospective students. A review of the contents of these letters by Prof. W. S. Rodman, of the University of Virginia, is presented below, briefly, under the headings of the original questions.

"1. (*a*) With but three exceptions the work was found to be in general what had been expected, but in practically all cases it had proved more varied and interesting than had been imagined.
(*b*) Practically unanimous affirmative.
(*c*) Because of its varied character, its value in training for future service, its helpful personal contacts, and general broadening influence.
(*d*) With few exceptions present activity is looked upon as a stepping-stone to better opportunities in the future; most of the writers visualize administrative positions as their goal.
(*e*) Practically unanimous affirmative.

"2. (*a*) Unanimously affirmative for those truly interested and having the necessary scholastic background and training.
(*b*) Fundamental subjects of theoretical character, particularly those that require supervision and that teach to think. More specifically, mathematics through differential equations, physics, mechanics, English, public speaking, economics, and a sprinkling of "cultural" and business courses. The indicated need for more emphasis on English is outstanding and it is rather significant that but one of the writers felt that a modern language other than English was at all essential in the engineering course.

"It appears from this group of letters that young engineering graduates normally find their activities interesting, valuable, and promising when associated with a company of the standing of the General Electric Company. It furnishes definite evidence that the engineering course in technical schools should emphasize those fundamental subjects which have recently been stressed in engineering education and that specialization should perhaps be postponed to post-graduate courses or to industrial activity itself."

Many Cities in Traffic Safety Contest

ALL BUT THREE cities having over 250,000 population, and three-fourths of all cities of more than 100,000 have entered the 1932 National Traffic Safety Contest. This is being sponsored by the National Safety Council through Sidney J. Williams, M. Am. Soc. C.E., who is director of its Public Safety Division and authority for the statements here given.

The purpose of the contest is to encourage interest among American cities in proper engineering attention to traffic problems, and specifically to lessen the steadily increasing number of traffic accidents. In 1931 these were responsible for a total of about 33,000 fatalities and, in addition, for injuries to approximately

one million people. They caused an estimated property damage of more than $900,000,000. The plan of the contest was first announced in December of last year. All cities of over 1,000 population were invited, without obligation or cost to themselves, to enter the contest, which will be judged on individual traffic-accident records and traffic safety programs and accomplishments for the entire year of 1932. Suitable "Safety City" awards will be presented by the national Safety Council to winners of the contest in each of the seven groups represented within the population limits of 1, 10, 25, 50, 100, 250, and 500 thousand and above.

There had been a remarkable response up to March 7. A total of 424 cities above 1,000 population had entered the contest, representing 54.5 per cent of the total population of all cities above 1,000, and more than 25 per cent of the population of the United States.

The following grading schedule will be used:

Accident records	50 points
Accident reporting	5 points
Traffic engineering	10 points
Traffic law enforcement	15 points
Child safety	10 points
Public education	10 points
Community safety organization	10 points

The awards will be made by a committee of nationally recognized experts headed by Dr. Miller McClintock, Director of the Albert Russel Erskine Bureau for Street Traffic Research, Harvard University.

American Arbitration Association Extends Its Service

IN COOPERATION with the American Chamber of Commerce in London, the American Arbitration Association has established a New York and London Commercial Arbitration Service for the benefit of those engaged in foreign trade. The president of the American Arbitration Association, Lucius R. Eastman, states that the recent abandonment of the gold standard by Great Britain, the imposition of high tariffs on imported goods, and the consequent possibility of increased differences arising out of contracts covering Anglo-American trade, make this extension of arbitration service particularly useful at this time. Special panels of arbitrators are being selected from among those who are familiar with trade relations between the United States and Great Britain.

A record of 2,567 controversies submitted by disputants for adjudication in the American Arbitration Tribunal, with the confirmation of the arbitrators' awards opposed in the higher courts in only ten instances and every disputed award upheld by the courts, is one of the achievements noted in the report of the American Arbitration Association, just issued, covering the six years during which the Association has been in existence. Matters arbitrated have covered nearly every kind of commercial dispute, and the amounts involved have reached several millions of dollars.

Identifying the "Explorer"

FROM C. N. Perry, M. Am. Soc. C.E., consulting engineer of Los Angeles, has come an account of an unusual discovery. To give the story in his own words:

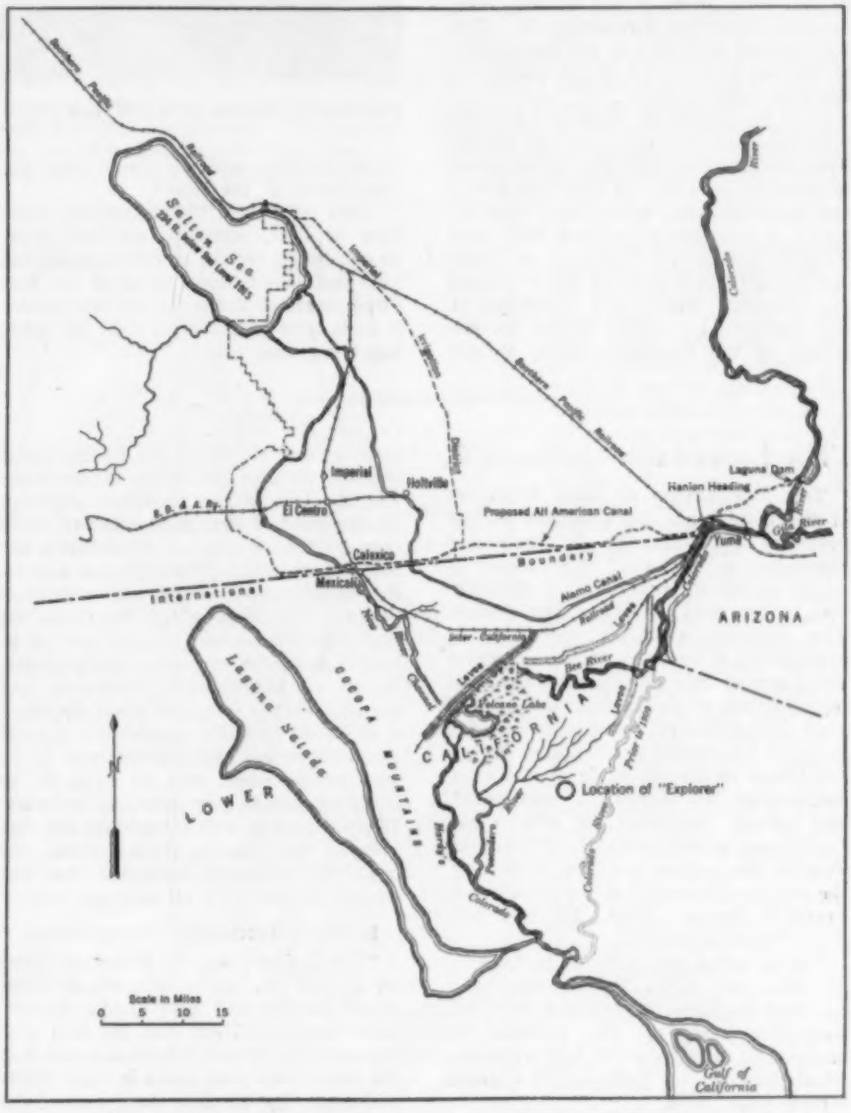
"The natural place in which one might expect to run across a boat is in the vicinity of some waterway. We were therefore somewhat surprised when, in the early part of 1930, while running a preliminary line for a canal through the dense growth which covers the delta of the Colorado River, our clearing came out at the steel hull of a boat.

"We realized that we were not the original discoverers, for the sand had been removed from around the hull. The delta is practically uninhabited, but later we ran across an old Mexican whose family had run cattle through that country for the last eighty years and he told us that his father had taken him to see this boat when he was about eight years old. As he was seventy-five years of age when we talked to him it would appear that the boat had been there prior to 1864.

"Probably when this man first saw the boat it was equipped with engine and boiler but these had since been removed and, either accidentally or intentionally, all the woodwork has been burned so that now but one small piece of the oak coaming is left. The bow, not having been buried as deeply in the silt, is in a good state of preservation but the remainder of the hull is so corroded that the shell can be ground to 'flour' between one's thumb and finger."

"EXPLORER" IS IDENTIFIED

"This boat is, without doubt, what is left of Lieutenant Ives' *Explorer*, which he had built for his exploration of the Colorado River in 1857 and 1858. The dimensions check in all respects and the history of the delta is sufficiently well known to justify the statement that no boat of this description, other than that of Ives, ever sailed on the Colorado. The *Explorer* now lies in Lower California, about fifty miles below Yuma,



COLORADO RIVER DELTA, SHOWING LOCATION OF THE "EXPLORER"

Ariz., and about two miles west of what was the channel of the Colorado until 1909, when the river changed to the westerly side of the delta. No doubt the boat broke loose from its moorings at some time when the river was in flood and was swept across country to its present location.

"Lieutenant Ives' description of this boat is to be found in his 'Report upon

THE "EXPLORER" UNDER FULL STEAM ON THE COLORADO RIVER, 1857
Chimney Peak in the Background

the Colorado River of the West,' explored in 1857 and 1858, 36th Congress, First Session, Executive Document 90. This was printed in 1861 and the frontispiece of the report is a full length picture of the boat under steam.

"For the benefit of those who may be interested but who may not find the report conveniently accessible, the following information as given by Ives concerning the boat has been taken from that report. It was ordered in June 1857 from Reaney, Neafie and Company of Philadelphia and was shipped to San Francisco via Panama. From San Francisco it was brought by sailing vessel to the mouth of the Colorado, where it was assembled. The hull was shipped in eight sections.

"Ives further describes the *Explorer* as being 54 ft. from the extremity of the bow to the outer rim of the stern-wheel. Amidships, the hull is left open like a skiff, the boiler occupying one-third of the vacant space. At the bow is a little deck on which stands the armament, a 4-lb. howitzer. In front of the wheel, another deck, large enough to accommodate the pilot and a few of the surveying party, forms the roof of a cabin 8 ft. by 7. The general appearance of the boat, while in commission, is shown

PRESENT CONDITION OF THE WRECK (1930)

in the drawing, which is traced from the frontispiece of the report.

"Ives completed his exploration some time in 1858, when he returned East. In closing his report, he refers to turning over this boat to some agent of the War Department at Yuma, so, without doubt, it must have broken away from its moorings soon after that."

The Loss of the "Jefferson"

THE ASSOCIATION of Field Engineers of the U.S. Coast and Geodetic Survey publishes semi-annually, in June and December, a "Bulletin," the purpose of which is the dissemination of information among field officers of the bureau. This bulletin, a unique and pleasing publication, 8 by 10 in. in size, contains 125 pages of information, drawings, and reproductions of photographs on the field work of the Survey. Although it is not prepared for general circulation, a limited number of copies are made available for distribution to engineers, navigators, and others interested in this work. Copies may be obtained without charge as long as the supply lasts by addressing the secretary-treasurer of the association, Frank S. Borden, Assoc. M. Am. Soc. C.E., Washington, D.C.

The following article by R. R. Lukens, M. Am. Soc. C.E., Hydrographic and Geodetic Engineer, is reprinted from the December issue of the Bulletin. It narrates an early failure to take a paddle-wheel steamer to California for coast survey work there.

"Of the many trials to get the much needed coast survey of the Pacific Coast started, perhaps the most disheartening was the loss of the steamship *Jefferson* on the coast of Patagonia while en route from Philadelphia to San Francisco in the summer of 1851. The *Jefferson* was an iron-paddle-wheel steamer with a draft of about 11 ft. New boilers were installed and other repairs were made to put her in condition for the long voyage through the Straits of Magellan to California. At her trials on the Delaware River she made a speed of 10 miles against the current and wind with a steam pressure of 25 lb. Her paddle wheel was 20 ft. 8 in. in diameter, and at easy steaming, she made 13 r.p.m., while with forced fires she was able to turn up 20 r.p.m. From the trials the engineers estimated that she would burn 5 tons of coal per day.

EARLY DIFFICULTIES ENCOUNTERED

"The *Jefferson* left the Delaware Capes on March 10, 1851, and struck head winds for the first part of the voyage. They soon found out that the coal consumption was 8 tons instead of 5 and that she made very poor speed in head winds and seas. By the time they reached the northeast trades, there were but 18 tons of coal left, so they shut down the plant and used sail. She had three masts and was probably barkentine rigged. While sailing, the buckets were removed from the paddle wheels to avoid their dragging and steam was raised only when in a calm. She arrived at Rio de Janeiro on April 24, 1851. . . .

"The *Jefferson* arrived at her next port, Montevideo, on May 4. Although no one on board had contracted yellow fever, they were subjected to a 6-day quarantine at this port. As they were now entering the season of winter gales, they experienced a series of storms while trying to coal from lighters, and it was not until May 19 that they sailed from this port. On May 24, during a very low barometer, a northerly gale overtook them and the *Jefferson* scudded before it with full steam and all sail that could be safely carried. On the afternoon of the 25th, a heavy cross sea boarded her and threw the vessel on her beam ends when she immediately broached to and lay exposed to the terrific seas following her.

"Orders were given to let fly all sheets (except the staysail), but finding that she did not recover, the order was given to cut away the masts. The crew jumped to this dangerous work with a will and succeeded in getting the fore and main over the side. Relieved of two of her three masts, she righted and fell off before the wind much to the delight of all hands, for while the masts were being cut away it seemed that she could never remain afloat. They were now able to heave to under steam and a reefed spanker, the only sail left.

"About 6 p.m. another heavy sea broke on board and swept her fore and aft, staving in the bulkheads of the engine room, wheelhouse, etc., and carrying the steering wheel overboard. This shock strained the hull terribly, and Murray [Lieutenant Commander F. K. Murray, the commanding officer] was afraid that she would break in two. As soon as the hatches could be opened, provisions, stores, and everything that could be spared were jettisoned and some improvement was noted. To add to the seriousness, the hull began to leak badly and the bilge pumps promptly choked up with coal and would not work. The frames of the engines had started from the hull and were momentarily expected to break. Under full steam they could make but 6 r.p.m. At this point, Murray states: 'It appeared that nothing but a miraculous interposition of Providence could save us, and we have great cause for gratitude that this was accorded us; for in a few minutes after the shock which disabled the vessel, the wind hauled to southward and began to die away, and the sea to subside.'

"They laid to until noon of the next day, when they found by observation that they were 110 miles northeast by north from Sea Bear Bay on the east coast of Patagonia. The sea having gone down, a jury mainmast was rigged and they painfully made their way to Sea Bear Bay, where they arrived on the afternoon of May 28, having taken over two days to make 110 miles. On June 2, Murray moved the vessel to Port Desire (within

sight of Sea Bear Bay), finding it better protected from the gales which prevail at this season.

"Murray sadly realized that the *Jefferson* was a hopeless wreck. She leaked badly, two of her masts were gone and her hull was badly hogged. He appointed a board of officers to survey the vessel and make recommendations. The board found her entirely unseaworthy and beyond repair except in a dry dock. As there were no such docks available, they recommended that, after removing all valuable property, the *Jefferson* be abandoned. This recommendation was approved by the commanding officer and the work of salvage was started at once.

"As ships were scarce, Murray chartered the first available vessel, a French barque, the *Austide*, of 160 tons, to carry his crew and the lighter machinery back to the United States. The *Austide* put into Montevideo for supplies, and there Murray and his crew transferred to the Naval supply ship *Relief*, which vessel arrived in New York November 2, 1851. The boilers, which were too heavy for the *Austide*, were shipped by a later vessel."

Pennsylvania Provides Employment at Pymatuning

ACCORDING TO information furnished by Edwin K. Morse, Director, Am. Soc. C.E., Engineer Member of the Water and Power Resources Board of Pennsylvania, the acute unemployment situation in that state is being relieved in part by a large state construction project, known as the Pymatuning Reservoir Project. This reservoir will fill a long-felt need. In fact, the beginnings of the project date back to the flood of 1907, which caused damage amounting to six million dollars in the city of Pittsburgh and its vicinity.

As a result of this catastrophe the leading engineers of the city banded together and gave their services for the purpose of making a study of the problem. The Pittsburgh Flood Commission was formed, and its engineers began to devise means of protecting the city from high water. They soon realized that efforts should be concentrated on work for flood prevention rather than for flood protection. Thus the problem assumed state-wide importance and was studied in detail by the Water Commission of the Commonwealth of Pennsylvania.

In 1914 this commission published a ten-volume report, which contained an inventory of the water resources of the state and outlined a comprehensive policy for the regulation of all the streams and lakes within its boundaries. Among the chief projects recommended in the report was the construction of a dam, the Pymatuning Dam, to form a huge lake at the source of the Shenango River, in the northwest corner of the state. A short distance below Newcastle the Shenango River joins the Mahoning River to form the Beaver River, which flows into the Ohio about twenty-five miles below Pittsburgh.

NEED FOR MORE WATER

The Shenango Valley is a highly industrialized district in which there are a great number of steel manufacturing plants. During the past twenty years there has been an acute scarcity of water in this section, especially in the late summer and fall months. During periods of drought, the same river water has been used at least 18 times in 24 hours by the steel mills, causing the temperature of the water to reach 140 deg. fahr. Also, navigation has been greatly retarded. No further industrial development is possible in the district until more water is available.

In 1913 the Pennsylvania State Assembly passed an act initiating the Pymatuning Reservoir Project, and since that date it has appropriated funds from time to time for the purchase of the land required. But it was only after the appropriation of $1,500,000 at the last session of the assembly that actual construction on the earth dam could be started. The present low cost of construction work has made a large part of the appropriation available for clearing, and for necessary changes in highway location and railroad alignment.

UNEMPLOYMENT SITUATION RELIEVED

In order to relieve the unemployment situation, Governor Pinchot has authorized the State Water and Power Resources Board, which has direct charge of the work, to proceed with construction on a day-labor basis. Tents and complete commissary equipment were set up at the site and in February 1932 the first two camps were opened, caring for a total of 800 men. Many more, for whom tents are not available, will be housed in nearby towns and transported to the work in trucks. They will be served with hot noonday meals, similarly transported.

When a man applies for work on the Pymatuning Reservoir Project he is registered, given a medical examination, and assigned to sanitary quarters. During the winter months 1,500 men have been cared for in this way, with the result that the acute distress in the northwest counties of the state has been relieved. More than 10,000 men have applied for employment on the project. Common labor receives 35 cents an hour for a nine-hour day, and food and lodging are furnished at cost, now 85 cents a day, but soon expected to be 75 cents and possibly 60 cents. The work now under way is largely confined to clearing, which will be completed by summer. Construction on the dam and on the highways and railroads will continue throughout the year 1932

LOCATION OF PROPOSED PYMATUNING RESERVOIR

LARGE LAKE CREATED

The Pymatuning Reservoir will flood 16,400 acres of land and will have a shore line 70 miles long. It will be longer than Lake Chatauqua and cover approximately twice as much ground as the Ashokan Reservoir although its capacity will be only about half as much as that of the latter, or about 64,275 million gallons. It is estimated that after construction of the dam the minimum regulated low flow of the Shenango River will be at least 400 sec-ft., that is, 50 times the extreme low-water flow of 1930. Delays and damage due to floods in the valley will be greatly reduced. The dam is to be 2,400 ft. long and constructed as an earth embankment with a concrete spillway. Its maximum height will be 50 ft., which will provide a 12-ft. freeboard above the normal reservoir level. According to present plans, the Pymatuning Reservoir Project will cost about three million dollars when completed. The state-owned shore property is to be converted into a public recreation park. It is also planned to stock the lake with fish and plant it with wild duck food. A wild life sanctuary of 4,700 acres will also be established at one end of the lake.

National Committee to Study Street Maintenance Economics

DURING the recent Road Show of the American Road Builders' Association, held in Detroit, representatives of nine nation-wide organizations met to organize a committee to study the economics of street maintenance work. This National Committee on Street Maintenance Economics expects to provide engineers with standard record forms and information as to the accounting and other administrative devices that should supplement a good organization and technical ability if effective street maintenance work is to be accomplished. Standard methods will be formulated to control street maintenance operations, to measure the amount of work done and its cost, to keep records of expenditures, and to prepare work programs and budgets.

The laboratory method of testing proposed procedures will be used. Cities of different sizes and of varying types will make experimental use of the methods under consideration in order to discover whether or not they are practicable. The committee is staffed and financed jointly by the American Road Builders' Association and the Research Committee of the International City Managers' Association. The official representative of the American Road Builders' Association, George B. Sowers, M. Am. Soc. C.E., was chosen chairman of the committee, on which the Society's representative is J. E. Root, M. Am. Soc. C.E. Other members of the Society on the committee are H. M. Waite and Robert B. Brooks.

Planning and Building Washington, D.C.

ONE OF THE able contributions to the Washington Bicentennial Celebration honoring our first president, who also achieved distinction as an engineer, is being made by the Washington Society of Engineers in the form of a book to be known as *The Planning and Building of Washington*. This work has been prepared by 17 writers distinguished in their respective fields. Besides Dr. F. H. Newell, under whom it is being edited, several contributors are members of the Society, including Col. U. S. Grant 3d, Assoc. M. Am. Soc. C.E., and J. B. Gordon, M. Am. Soc. C.E.

It is intended not only to use the work as a contribution to the Bicentennial Celebration but also to distribute it widely and sell it for many years as an authoritative reference work on the nation's capital. Special attention is given in the volume to those features of the city that are of interest to engineers and architects. In addition emphasis is placed on the activities of George Washington as a city planner in initiating the work of outlining and building the great capital of a great nation.

The Washington Society of Engineers includes engineers in all branches of the profession in and around the Capital. Many of its active supporters are members of the Society, including A. B. McDaniel, its president for the current year. The Washington Society believes that this volume should be in the library of every engineer. Copies may be secured through its office or directly from the publishers, Ransdell, Inc., 810 Rhode Island Avenue, N.E., Washington, D.C.

NEWS OF ENGINEERS

From Correspondence and Society Files

ARTHUR J. TRAPP has been recently appointed Assistant Construction Engineer under the Supervising Architect of the Treasury Department, in Boston, Mass. Formerly, Mr. Trapp was Assistant Engineer-Foreman, Department of Commerce, Lighthouse Service in Portland.

CHARLES N. STRONG is associated with the Lautaro Nitrate Company, Ltd., in Tocopilla, Chile.

EDWARD H. DEETS has been appointed Right-of-Way Examiner in the Right-of-Way Department of the Maryland State Roads Commission and will have headquarters at Frederick, Md. For the past 9 years, Mr. Deets has been Associate Engineer with the C. N. Harrub Engineering Company and its predecessors, Klyce and Harrub and B. H. Klyce, Municipal Consulting Engineers of Nashville, Tenn.

WILLIAM N. HALL is now in the Reproduction Plant, office of Chief of Engineers, War Department, Washington, D.C.

ROY E. EVERETT, heretofore Designer for the Allied Engineers, Inc., in Jackson, Mich., is now connected with the Michigan State Highway Bridge Department in Lansing.

JASPER H. INGS, formerly with the Beauharnois Construction Company, Quebec, Canada, is now in the Department of Mechanical Engineering, Faculty of Applied Science and Engineering, University of Toronto.

HUBERT W GREENE is connected with the Louisiana Highway Commission in Natchitoches, La.

ARTHUR W. BARRUS, Estimator and Construction Engineer for the Frederick Page Contracting Company, Inc., in New York City, was formerly connected with the William J. Taylor Company, also in New York.

HUGH JONES is now in Los Angeles where he is connected with the Metropolitan Water District of Southern California.

ERNEST W. SEEMANN, Construction Engineer for the Merritt-Chapman and Whitney Corporation in Duluth, was formerly associated with the Portland Cement Association in Minneapolis, as Field Engineer.

HYMAN H. CASHDAN has accepted a position with M. Shapiro and Son, Engineers and General Contractors in New York City.

IVAN NELIDOV, Senior Engineer of Hydraulic Structure Design, State Department of Public Works, in Sacramento, was previously engineer for the State Division of Water Resources.

STANLEY PINEL is now a consulting engineer with Griffenhagen and Associates, Management Engineers, in Hartford, Conn.

JAMES H. T. MCGEE has been promoted to Office Engineer, Barron G. Collier Interests, Everglades, Fla.

MAXCY R. HALL is now engaged in private engineering practice in Atlanta, Ga., associated with B. M. Hall and Sons.

JOHN R. FORDYCE, formerly a consulting engineer in Hot Springs National Park, Ark., has recently been appointed joint engineer, with CLIFFORD OLDER of Chicago, for the purpose of checking the engineering work of the Arkansas State Highway Commission.

PHILIP W. KNISKERN has been appointed President of the Property Ownership Corporation of New York City, with offices at 17 East 42d Street. His former position was with the Continental Mortgage Company, as Vice-President and General Manager.

H. J. FLAGG, Commercial Engineer for the Federal Light and Traction Company in New York City, was previously Assistant to the General Manager of Grays Harbor Railway and Light Company in Aberdeen, Wash.

J. A. GUTTRIDGE has been transferred to the Watershed Department of the New York Board of Water Supply as Department Engineer.

W. C. BROCKWAY, formerly Special Representative for the Johns-Manville Sales Corporation in Boston, is now in Montreal where he is associated with Building Products, Ltd.

JOHN BERG is now State Engineer of South Dakota.

ALFRED J. CLEARY has been appointed Chief Administrative Officer of San Francisco. He will supervise eleven of the San Francisco city departments and will operate them under the provisions of the new city charter.

WARREN C. ACKERSON is now associated with United Engineers and Constructors in Philadelphia.

Changes in Membership Grades

Additions, Transfers, Reinstatements, Deaths, and Resignations

From February 10 to March 9, 1932

Additions to Membership

ABBATE, ALFONSO (Jun. '31), 6063 Palmetto St., Brooklyn, N.Y.

ALBERT, ODD PER LEIF (Assoc. M. '32), 321 Summit Ave., Jersey City, N.J.

ALLISON, LIMUEL RILEY (Assoc. M. '32), Gen. Contr. (Everly & Allison), 900 North 1st St., Albuquerque, N.Mex.

ANDEREGG, RUPERT ANDREW (M. '32), Associate Prof., Civ. Eng., Univ. of Cincinnati (Res., 3607 Bellecrest Ave.), Cincinnati, Ohio.

ANDREWS, EARLE Topley (Assoc. M. '32), Engr., Pennsylvania Glass Sand Corp., Lewistown, Pa.

ANNICK, WILLIAM ALFRED (Jun. '32), Levelman, Bureau of Eng., care, City Engrs. Office, Scranton, Pa.

BABB, BENJAMIN MCKINLEY (Assoc. M. '32), City Engr., City Engrs.' Office, Lawrence, Kans.

BENTON, KELLY EDWARD (Jun. '31), 440 U.S. Custom House, Denver, Colo.

BOWERS, MARVIN LAYAFETTE (Assoc. M. '32), Res. Engr., State Highway Dept., Box 663, Corsicana, Tex.

BRADY, WILLIAM VEIRS (Assoc. M. '32), Civ. Engr., Box 235, Garden Grove, Calif.

BROWN, CHARLES VINES, JR. (Jun. '31), 1332 Chamber St., Vicksburg, Miss.

BURTON, THOMAS EDWIN (Assoc. M. '32), Div. Eng., 1st Dist., State Highway Comm. (Res., 1260 Plass Ave.), Topeka, Kans.

CASSIDY, GEORGE FRANCIS (Jun. '31), Eng. Asst., Board of Transportation, 250 Hudson St. (Res., 318 East 136th St.), New York, N.Y.

CHANDLER, JAMES FREDERICK (Jun. '31), U.S. Eng. Office, Paducah, Ky.

CHERNO, JOHN (Assoc. M. '31), 584 Thirty-Fourth St., Sacramento, Calif.

COHEN, CHARLES (Jun. '32), Asst. Engr., Dept. of City Transit (Res., 6213 Delancey St.), Philadelphia, Pa.

CRAIG, HOWARD RAYMOND (Jun. '31), 404 East Auglaize St., Wapakoneta, Ohio.

DEAN, WILLIAM ENNELS, JR. (Jun. '31), Instrumentman, State Highway Comm., Church Point, La.

EARL, RALPH (M. '32), Civ. and Mech. Engr. (Earl Eng. Co.), Room 2002, Am. Bank Bldg., New Orleans, La.

FERNANDEZ, RODOLFO MAXIMILIANO (Jun. '32), Ave. Uruguay 35, City of Mexico (D.F.), Mexico.

FLETCHER, PHILIP NELSON (Jun. '32), Eng. Materials Laboratory, Univ. of California, Berkeley, Calif.

FORST, MERLE ALFRED (Jun. '32), 3103 Fourth Ave., Beaver Falls, Pa.

GALINATO, JUAN GALINATO (Jun. '31), 49 Greenwich Ave., New York, N.Y.

GILMORE, FRANK SEVERT (Assoc. M. '32), Asst. State Maintenance Engr., State Highway Comm. (Res., 1612 West 6th St.), Topeka, Kans.

GUPPY, JOHN WARREN (Assoc. M. '32), Asst. Engr., Am. Bridge Co., 30 Church St., New York, N.Y. (Res., 165 Franklin St., Bloomfield, N.J.)

HALDANE, JOHN NAFTEL (Jun. '31), Draftsman, New England Fuel & Transportation Co. (Res., Prescott House), Everett, Mass.

HAMILTON, VIVIAN EARLE JR. (Assoc. M. '31), 619 Rusk Bldg., Houston, Tex.

HARRIS, WALTER J. (Assoc. M. '32), Civ. Engr., Grade IV, Los Angeles County Road Dept., 1202 Hall of Records, Los Angeles, Calif.

HART, ROBERT WINSTON (Jun. '31), 1419 Division St., Key West, Fla.

HEAD, JAMES RAYMOND (Assoc. M. '32), with California Corrugated Culvert Co., West Berkeley (Res., 1100 Esplanade, Chico), Calif.

HENDON, HARRY HOLMAN (Jun. '32), Asst. Engr., Jefferson County Eng. Dept., Room 216 Court House, Birmingham, Ala.

HERKOMER, JOHN ANTHONY (Jun. '32), Instrumentman and Asst. Engr., Pacific Gas & Elec. Co., Salt Springs, Martell, Calif.

HESS, HARRY LEE (M. '32), Engr., Ralph A. Beebee, 1 Montgomery St., San Francisco (Res., 748 Mandana Boulevard, Oakland), Calif.

HIRSCHFELDER, JOHN (Jun. '32), Draftsman and Field Engr., L. S. Rosener, 2280 Pacific Ave., San Francisco, Calif.

HOCHSTEIN, IRWIN (Jun. '32), Designer, Purdy & Henderson, 45 East 17th St., New York (Res., 1654 East 3d St., Brooklyn), N.Y.

HOLDAMPF, CARL RICHARD (Jun. '32), Civ. Engr., C. S. Whitney (Res., 3465 North Humboldt Ave.), Milwaukee, Wis.

HUFFINE, WILLIAM BYRD (Jun. '32), Junior Highway Engr., Div. of Management, Bureau of Public Roads, Washington, D.C.

HUGHES, WILLIAM FRANCIS (Jun. '31), 123 G St., Salt Lake City, Utah.

HUIE, ALBERT VAN ARNAM (Jun. '31), Gen. Contr. (Irving V. A. Huie, Inc.), Saugerties, N.Y.

INTEMANN, HENRY LUTHER (Jun. '32), Acoustical Engr., Rocky Mountain Celotex Co. (Res., 1544 Adams St.), Denver, Colo.

ISAACSON, GEORGE HAROLD (Jun. '31), 70 Villa Ave., Yonkers, N.Y.

JOHNSON, JOHN ALBERT (Jun. '32), Engr. Asst., Grade 3, Board of Transportation, New York (Res., 725 Fourth Ave., Brooklyn), N.Y.

KENNY, EDWARD FRANCIS (Jun. '31), Transitman, Grade 4, Dept. of Parks, Brooklyn (Res., 214 East 203d St., New York), N.Y.

KLEIN, MICHAEL (Jun. '32), Junior Engr., Board of Transportation (Res., 101 Second Ave.), New York, N.Y.

LANGBEIN, WALTER BASIL (Jun. '32), Constr. Engr., Rosoff Subway Constr. Co. (Res., 2385 Grand Concourse), New York, N.Y.

LARSON, CLARENCE EDWARD (Jun. '31), 12655 Sussex Ave., Detroit, Mich.

LENZ, ARNO THOMAS (Jun. '32), Instr., Hydr. and San. Eng., Univ. of Wisconsin, Hydr. and San. Laboratory, Univ. of Wisconsin, Madison, Wis.

LOSEE, LAWRENCE KNICKERBOCKER (Jun. '32), Upper Red Hook, N.Y.

LUFF, WILLARD JOHN (Assoc. M. '32), Cons. Engr., 3046 Lincoln Boulevard, Cleveland Heights, Ohio.

MAAG, ERNST (Assoc. M. '32), Engr., Bldg. Dept., City of Pasadena (Res., 809 Summit Ave.), Pasadena, Calif.

MCNATT, EDMUND ALSTON (Jun. '31), Box 183, Tivoli, Tex.

MATHIS, ALTON PARKER (Jun. '31), 30 Astor St., Newark, N.J.

MERRILL, KEITH BARTLETT (Assoc. M. '32), Cons. Engr., Southern Surety Co. of New York, 712 Southern Surety Bldg., Des Moines, Iowa.

MOSER, CECIL RAMSEY (Assoc. M. '31), 40 South McLean St., Memphis, Tenn.

NOALL, JAMES WESLEY (Jun. '31), 327 Third Ave., Salt Lake City, Utah.

NOLAND, THOMAS JEFFERSON, JR. (Jun. '31), care Hoover Dam Cooperative Investigation, 202 Eng. Materials Laboratory, Univ. of California, Berkeley, Calif.

PAIGE, EMMETT LESLIE (Jun. '31), 20 Clinton Pl., Metuchen, N.J.

PATELLA, GIUNIO (Jun. '32), Transitman, Allegheny County (Res., 341 Ladson St., E.E.), Pittsburgh, Pa.

PEARSON, JAMES THOMAS MORLEY (M. '32), Chf. Engr., Shell Eastern Petroleum Products, Inc., 122 East 42d St., Room 1518, New York, N.Y.

PEOPLES, HAROLD DENTON (Jun. '31), 1016 South 4th St., Leavenworth, Kans.

PETERSEN, RUSSELL WALLACE (Jun. '32), Route 1, Gowen, Mich.

PREECE, EDMUND FRANCIS (Assoc. M. '32), Designing Engr., Ferruccio Vitale, 101 Park Ave., New York, N.Y. (Res., Silvermine Falls, Norwalk, Conn.)

REANTILLO, TEOFILO PASCUA (Jun. '31), Bridge Draftsman and Designer, State Highway Dept., 921 Bergen Ave. (Res., 19 Bryan Pl.), Jersey City, N.J.

RODRIGUEZ Y CATARROJA, FILEMON (Jun. '31), 17 Gomez St. Addition, San Juan, Rizal, Philippine Islands.

SALING, EARL FRANKLIN (Jun. '31), care, State Highway Engrs., Banks, Ore.

SAMSON, GERALD ALBERT (Jun. '31), Junior Engr., U.S. Bureau of Reclamation (Res., 1933 Sherman St.), Denver, Colo.

SCHANTZ, MAXWELL (Assoc. M. '31), 4824 Church Ave., Brooklyn, N.Y.

SEDGWICK, ALLAN ERNEST (M. '32), Geologist and Cons. Engr., 608 South Hill St., Room 900, Los Angeles, Calif.

SLEEGER, WARREN HACKMAN (Jun. '32), Asst. Engr., Met. Drainage Comm. of Minneapolis and St. Paul, 205 Oak Grove Pl., St. Paul, Minn.

STATHAM, WILLIAM HAROLD (Assoc. M. '32), care, Eastern Vibro Concrete Pile Co., 152 Passaic St., Newark, N.J.

STEVENS, CHARLES JAMES (Jun. '31), 1234 Vance Ave., Coraopolis, Pa.

TULL, ROBERT ALEXANDER (Jun. '31), Office Engr., State Highway Dept., Box 95, Paducah, Tex.

TURNER, HENRI BATCHELDER (Jun. '31), 16 Forest Ave., Everett, Mass.

TWIGGAR, EDWARD VERNON (Jun. '31), 307 Lincoln Ave., Charleroi, Pa.

WALKER, JAMES DONALD (Jun. '32), San. Engr., The Am. Well Works (Res., 705 Penn. Ave.), Aurora, Ill.

WEIGAND, EDWIN JOHN (M. Feb. '32), Senior Engr., Commrs. of Sewerage, 400 Marion E. Taylor Bldg., Louisville, Ky.

WELDEN, RICHARD WAYNE (Jun. '32), Supt. of Constr., Welden Bros. (Res., 710 Union St.), Iowa Falls, Iowa.

WILDE NORTON CHARLES (Jun. '32), with Guy Vroman, Larchmont (Res., 3312 Giles Pl., New York), N.Y.

WILSON, JOHN WADSWORTH (Jun. '31), 7916 Oak St., New Orleans, La.

WOODYARD, FRANK ALBERT (Assoc. M. '32), (J. F. Woodyard, Jr., & Son), Apartado 397, Monterrey (N.L.), Mexico.

YOUNGS, LYMAN GUSTIN (Jun. '31), Box 166, Moscow, Idaho.

ZEPP, PAUL LATIMER (Jun. '31), Apprentice Asst., Track Supervisor, B. & O. R. R., 21 East B St., Brunswick, Md.

Membership Transfers

CARTER, HAROLD SAMUEL (Jun. '24; Assoc. M. '27; M. '32), Cons. Engr.; Prof. and Head Civ. Eng. Dept., South Dakota State Coll. (Res., 807 Eighth St.), Brookings, S.D.

FERGUSON, PHIL MOSS (Jun. '26; Assoc. M. '31), Associate Prof., Civ. Eng., Univ. of Texas, University Station, Austin, Tex.

HACKER, THEODORE WARREN (Assoc. M. '24; M. '31), Associate, Whitman, Requardt & Smith, 1602 Baltimore Trust Bldg., Baltimore, Md.

JOHNSON, WILLARD HAVEN (Jun. '26; Assoc. M. '32), Senior Eng. Draftsman and Asst. Engr., Transit Bureau, City of Newark, City Hall, Newark (Res., 794 Broad St., Bloomfield), N.J.

KERR, CLARENCE MARION (Jun. '30; Assoc. M. '32), Dist. Supt. of Constr., Granite Constr. Co. (Res., 920 North 1st St.), San Jose, Calif.

KOFFSKY, SAMUEL (Jun. '28; Assoc. M. '32), Asst. Engr., William Russell Davis, 1 Columbia Pl. (Res., 623 Myrtle Ave.), Albany, N.Y.

LEWIS, GEORGE DRAPER (Assoc. M. '25; M. '32), 825 Myrtle Ave., Albany, N.Y.

LYONS, WILLIAM THEODORE (Jun. '29; Assoc. M. '32), Gen. Contr., 3671 Broadway, New York, N.Y.

PALMER, CHESTER BONNEY (Jun. '25; Assoc. M. '32), Asst. Engr., Pacific Gas & Elec. Co., 245 Market St., San Francisco, Calif.

POPPER, MAX JOSEPH (Jun. '27; Assoc. M. '32), Asst. Engr., Dept. of Plant and Structures, 18th Floor, Municipal Bldg., New York, N.Y.

RICHARDS, LEON KENNETH (Jun. '27; Assoc. M. '31), Supt., Reed & Wheelock, 916 North 19th St., Waco, Tex.

WILLIG, WALTER LEE (Jun. '28; Assoc. M. '32), Bridge Designer, State Highway Comm., 921 Bergen Ave., Jersey City, N.J. (Res., 555 Westchester Ave., Port Chester, N.Y.)

REINSTATEMENT

BLIZNIAK, EUGENIUS BARTHOLOMEW, M., reinstated Mar. 8, '32.

RESIGNATIONS

AURYANSEN, WINSLOW CHAPMAN, Jun., resigned Feb. 19, '32.

CLELLAND, STANLEY HOLCOMB, Assoc. M., resigned Feb. 16, '32.

ENDO, TOKICHI, M., resigned Feb. 26, '32.

GREEN, JAMES RAYMOND, Assoc. M., resigned Feb. 10, '32.

HENDREN, ROBERT FULTON, Jun., resigned Feb. 17, '32.

POSEY, MASON ELI SAUNDERS, M., resigned Mar. 1, '32.

SCHROEDER, EDWARD ALBERT, Jun., resigned Feb. 9, '32.

SILSBEE, JAMES ALFRED, Assoc. M., resigned Feb. 12, '32.

SMITH, WILLIAM HENRY, Assoc. M., resigned Feb. 26, '32.

TOUCEDA, ENRIQUE AUGUSTO, M., resigned Feb. 29, '32.

DEATHS

ALGER, JULIAN WILMOT. Elected Assoc. M., Mar. 16, 1925; died Jan. 27, 1932.

BOARDMAN, HOWARD EDWARD. Elected Jun., Oct. 7, 1902, Assoc. M., Jan. 4, 1905, M. Oct. 1, 1912; died Feb. 28, 1932.

BUNN, PAUL KING. Elected Assoc. M., Mar. 16, 1925, M., May 25, 1931; died Jan. 23, 1932.

DAHM, SVERRE. Elected M., Feb. 7, 1906; died Feb. 14, 1932.

FERNSTROM, HENNING. Elected M., May 7, 1902; died Feb. 26, 1932.

KENNEDY, JEREMIAH JOSEPH. Elected M., June 4, 1902; died Feb. 15, 1932.

MENDIOLA, MANUEL MARIA. Elected M., Dec. 5, 1906; died Jan. 31, 1932.

OLDS, THOMAS HARTMAN. Elected Assoc. M., May 6, 1914; died Feb. 10, 1932.

SMITH, AUGUSTUS. Elected M., June 4, 1902; died Feb. 29, 1932.

SUCKOW, VERNON CHRISTOPHER. Elected M., Nov. 15, 1926; died Feb. 12, 1932.

VON GELDERN, OTTO. Elected M., May 4, 1892; died Feb. 17, 1932.

WALKER, WILLIAM THOMAS. Elected M., June 30, 1911; died Feb. 25, 1932.

TOTAL MEMBERSHIP AS OF MARCH 9, 1932

Members	5,863
Associate Members	6,355
Corporate Members	12,218
Honorary Members	18
Juniors	2,902
Affiliates	123
Fellows	5
Total	15,266

Men Available

These items are from information furnished by the Engineering Societies Employment Service with offices in Chicago, New York, and San Francisco. The Service is available to all members of the contributing societies. A complete statement of the procedure, the location of offices and the fee is to be found on page 97 of the 1932 Year Book of the Society. Unless otherwise noted, replies should be addressed to the key number, Engineering Societies Employment Service, 31 West 39th Street, New York, N.Y.

CONSTRUCTION

GRADUATE CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 30; married; 8 years experience, railroad, highway, and bridge work; construction and maintenance. Speaks Spanish fluently. Good recommendations. Location immaterial. Available immediately. D-484.

CONSTRUCTION ENGINEER; Jun. Am. Soc. C.E.; graduate of Massachusetts Institute of Technology; 5 years general building experience. Capable of handling office or outside superintendence for contractor or engineering firm doing construction work. Available at once. Will go anywhere. C-1487.

CONSTRUCTION EXECUTIVE; M. Am. Soc. C.E.; graduate of Harvard University; 15 years as general manager of engineering construction company designing and building industrial plants, bridges, water works, and commercial buildings. Desires district management with outstanding company where valuable contacts and proved business ability are required, or management of smaller company needing rehabilitation. D-148.

CIVIL ENGINEER; M. Am. Soc. C.E.; graduate, C.E. degree; registered in three states; 20 years experience in design and construction of steel and concrete bridges and structures. Last 6 years has had charge of over $1,000,000 worth of heavy construction each year for contractor and engineer. Available immediately. Location immaterial. B-6571.

GRADUATE ENGINEER; Assoc. M. Am. Soc. C.E.; 29; married; 5 years experience on construction of steam and hydro-electric power plants, substations, and transmission lines. Postgraduate work in structural engineering. Desires position with utility company or consulting engineer. Location immaterial. Available at once. C-9742.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; married; graduate of Rensselaer Institute; 6 months experience on stadia topographical surveys; 9 months as field engineer on pneumatic caisson foundations; 4 years on design, location, and construction of concrete highways and bridges; 5 years of railroad engineering including location, yard layouts, and general maintenance. Available immediately for field or office work in any location. D-259.

CONSTRUCTION SUPERINTENDENT, ENGINEER; Assoc. M. Am. Soc. C.E.; 39; married; 18 years construction experience on factory and office buildings, steel mills, gas plants, track work, hydro-electric and steam plants, and storm and sanitary sewers. Now available. Excellent references. Location immaterial. C-9526.

DESIGN

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 26; single; graduate of Massachusetts Institute of Technology, 1927; 5 years experience in all phases of design and construction of industrial buildings, field and office; 3 years experience in all phases of design and construction of airports. Majored and thesis in hydro-electric engineering. Desires work in field or office anywhere. D-448.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 27; M.S. (Illinois); graduate study here and abroad in structural engineering; 4 years experience in surveying, designing, and estimating reinforced concrete and steel structures; contractor's engineer, estimating, buying, writing contracts; speaks German fluently, French slightly. Location immaterial. C-7112.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; college graduate; 10 years experience; 6½ years with Westchester County Park Commission as structural designer. Experience on grade crossing elimination work, parkway projects, and similar work. Thoroughly familiar with skew arches, statically indeterminate structures, and Beggs deformeter gages. Familiar with research work and investigations. Can also do field work. B-6520.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 25; M.E., 1927; steel and reinforced concrete designer and draftsman; 3 years field and office experience for contractor on concrete dams and dock works, including construction plant; 2 years in design office on large bridge and tunnel projects. Speaks French, German, and Spanish. Any locality. D-457.

CIVIL AND STRUCTURAL ENGINEER; Jun. Am. Soc. C.E.; B.S. in C.E. and C.E. degrees; about 5 years experience in New York City as a designer, detailer, draftsman, checker, and estimator of steel and reinforced concrete. References and samples of work sent on request. C-6186.

PAPER MILL DESIGNER AND DRAFTSMAN; Assoc. M. Am. Soc. C.E.; 33; married; graduate civil engineer; 8 years experience in design and construction of pulp and paper mills (including complete equipment and piping layouts), steel, concrete, and timber structures. Naturalized. Speaks Russian. Available now. Location anywhere, but Pacific Coast preferred. D-119-311-A-7—San Francisco.

CIVIL AND STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; 32; married; university graduate; 10 years experience in designing reinforced concrete, steel, and timber structures both industrial and commercial. Familiar with steel and petroleum industry requirements; foundations and reinforced concrete a specialty. Connection with engineering firm desired. Available immediately. Best references. D-480.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; graduate of Johns Hopkins University; married; no children; 3 years experience in sanitary engineering field, particularly on design, operation, and control of sewage treatment plants. Available at once and will go anywhere. D-550.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 24; married; graduate of Clemson College; 3 years experience in highway bridge office doing tracing, detailing, and designing; present rating of detailer. Valuable experience in lumbering and construction work. Capable of field or office work. Available on short notice. D-555.

EXECUTIVE

MANAGER AND SUPERINTENDENT OF CONSTRUCTION; M. Am. Soc. C.E.; 37; married; university graduate; 15 years experience on all types of foundations, dams, hydro-electric developments, and various structures. Fully experienced organizer for cost and speed, contract or other work, domestic and foreign, as well as for planning and planting projects. Also experienced in estimating and designing. Available now. C-9694.

DREDGING ENGINEER; M. Am. Soc. C.E.; 20 years experience with three largest dredging companies. Thoroughly familiar with design, construction, and operation of all sizes of hydraulic and dipper dredges, subaqueous multi-frame rock drills, floating revolving cranes, and tug boats. Also has made investigations and analyses of dredging operations and prospective work. D-578.

EXECUTIVE ENGINEER; M. Am. Soc. C.E.; American; graduate; 25 years experience in manufacturing, management, finance, sales, construction of plants, installation of equipment, rehabilitation of plants, and economical handling of labor and materials. Experienced in the manufacture of engines, boilers, grey iron castings, hardware, structural steel, and textiles. C-1432.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 28; married; graduate; 6 years experience in office and field; capable of designing, estimating, and directing work; steel and concrete bridges, difficult foundations, scheduling, costs, and the efficient placing of labor. Location immaterial. Available immediately. D-479.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; married; technical graduate; 19 years experience in sanitary and municipal work embracing design and construction of sewerage systems, including treatment works, water supply systems, streets and pavements, preliminary investigations, and reports. Seeks connection with consulting engineering or contracting firm. A-466.

STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; 29; married; graduate of Rensselaer Polytechnic Institute; 9 years experience. Last 7½ years with consulting engineer, specializing in design details, specifications, and estimates of fixed and movable bridges; structural design on buildings. Especially proficient in design of vertical lift bridges. Past 2 years in charge of office. Location, East. Available May 1. D-506.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; single; 1924 graduate. Experience in precise surveying, design and construction of railway bridges, and with contracting firms as representative and superintendent. Prefers location in the South with good consulting or construction engineering firm, with salary secondary to future. Available on one week's notice. C-8381.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 33; married; excellent health; B.A. and B.E. graduate of University of Dublin, 1922; 5½ years highway and railroad bridge design; 3 years city planning; 6 months teaching mathematics. Available immediately. Will go anywhere in North America. Particulars and references on request. B-6280.

GRADUATE CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 8 years varied structural experience, steel and concrete design and construction, estimates, inspection, surveys, steel and track layouts, and related work. Capacity to assume responsibility. Desires connection with construction company, consultant, contractor, or architect in field or office; also sales-engineering or instructorship. C-2605.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 41; graduate of University of Wisconsin. Last 10 years with state highway organization as drainage engineer, assistant chief of location division, resident engineer on bridge construction. Preceding 9 years on topographic surveys, utility surveys, railroad location and construction, and municipal engineering. Available immediately. Location unimportant. D-537.

CIVIL ENGINEER; M. Am. Soc. C.E.; 20 years experience as construction superintendent, resident engineer, designer, surveyor, inspector, and draftsman. Position desired. Experienced in the design and construction of steam, hydro-electric, and cement plants, irrigation works, railroad structures, sewers, and pavements. High class references. D-535.

CONSTRUCTION ENGINEER; Assoc. M. Am. Soc. C.E.; 39; graduate civil engineer; 17 years thorough and practical experience as project manager on construction of public works, housing groups, commercial and industrial buildings. Experience covers design, estimating, purchasing, supervision, and expediting. Location immaterial. Available on short notice. Detailed record furnished on request. B-4964.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; married; 22 years active design and construction experience on railroads, bridges, water, and sewer systems. Experienced in supervision of office and field forces, preliminary investigations, design, contract plans, estimates, specifications, and construction. Location immaterial. Available immediately. D-545.

SPECIALIST IN BUILDING CONSTRUCTION; M. Am. Soc. C.E.; 25 years experience structural and architectural engineering and superintendence on all types of buildings. Past 4 years chief engineer for one of the largest general building contractors in the East. Available at once. Prefers to locate in the East, but will go anywhere. C-3918.

CIVIL AND INDUSTRIAL ENGINEER; M. Am. Soc. C.E.; graduate; licensed two states. Experience covers complete charge of design and construction, including preliminary investigations, studies, and estimates of industrial plants, housing groups for state institutions and private interests, and hydro-electric developments, including all utilities and architectural work in connection therewith. Licensed two states. B-2835.

JUNIOR

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 26; 6 years experience railroading, drafting and designing, road work, general construction estimating, steel and iron shop experience, electrification, and building alteration and maintenance. Desires position with contractor, builder, or consulting engineer, both in field and office. Location, metropolitan area or Washington, D.C. C-6821.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 22; single; graduate of Rensselaer Polytechnic Institute; experience with a civil and sanitary engineer doing office and field work, including work on field party, inspecting, drafting, and computing; recent experience in stadia surveying, computing, and plotting; desires civil engineering work. C-9813.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 23; married; civil engineering graduate, 1931; 6 months experience with state highway department. Desires work in any engineering capacity, but best fitted for work involving mathematics. Has excellent college record in both civil engineering and mathematics. References and college record sent on request. Available immediately. D-450.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; Affiliate, A.S.M.E.; 26; single; graduate of University of Illinois; passed U.S. Civil Service junior engineer examination. Desires position in any branch of civil engineering, preferably structural or hydraulic works. Salary secondary. Location anywhere in the world. D-392.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 1928 graduate, B.S. in C.E.; 1½ years on highway construction; 2 years on hydro-electric developments, including inspection, surveying, and office work. Desires any position within limits of training or experience. Location immaterial. C-7292.

GRADUATE CIVIL ENGINEER; Jun. Am. Soc. C.E.; 26; single; 2 years experience highway surveying and drafting; 2 years with public utility as surveyor and field engineer, selecting routes for new telephone lines and locating and engineering same. Salary secondary to opportunity to connect with responsible engineering firm. Available now. Location immaterial. C-6253.

STRUCTURAL ENGINEER; Jun. Am. Soc. C.E.; 23; graduate, 1930; 1¼ years structural steel detailing; just completed 4 months course in practical electric arc-welding and its application to designing steel structures at plant of leading company. Connection with Eastern structural firm using welding or planning to do so preferred. C-7243.

ARCHITECTURAL ENGINEER; Jun. Am. Soc. C.E.; 25; single; Bachelor of Arts, Physics Major, and Bachelor of Science, Architectural Engineering Major, at University of Colorado 1927 and 1929; Master of Science, Structural Engineering Major, University of Illinois, 1930. Accoustical engineer, Denver, Colo., from 1930 to 1931. Available immediately. Location anywhere. D-554—3111-A-5 San Francisco.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 22; single; recent graduate of University of Pennsylvania, 1931. Desires opportunity in traffic or transportation engineering, construction, or surveying. With railroad company during past two summers surveying. Available immediately. Location immaterial. D-547.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 23; single; graduate; 2½ months railroad survey preliminary to construction; 10 months experience in concrete testing laboratory on research and inspection at mixing plant and gravel plant. Can read and write Spanish. Small salary and expenses. Available immediately. Location foreign, but South America preferred. D-532.—323-A-1 San Francisco.

MUNICIPAL ENGINEERS

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 28; married; graduate; 6 years engineering experience on designs, plans, and estimates for highway bridges, including work on rigid frame; special highway studies; some municipal and park work. Able to plan and direct work. Desires position in one of the foregoing fields. Available immediately for location in the East. D-421.

SALES

STRUCTURAL SALES AND DESIGN; M. Am. Soc. C.E.; 44; thoroughly experienced in all departments of steel fabricating business. For past 15 years have directed the estimating, designing, pricing, and sales. Available immediately. C-5095.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 25; single; B.S., 1927; experienced in refractory and glass plant design; last 2 years on railroad bridge work and structural design and detailing. Desires position in structural or sales engineering capacity or teaching. Location anywhere. C-3256.

STRUCTURAL AND SALES ENGINEER; Assoc. M. Am. Soc. C.E.; 38; married; graduate. Over 12 years experience in sales and sales management. Experience in designing, detailing, and estimating of bridges and buildings, specializing in steel. Desires position with building material manufacturer or general contractor. Available immediately. Executive ability. Capable and forceful. D-16.

TEACHING

PROFESSOR IN CIVIL ENGINEERING; Assoc. M. Am. Soc. C.E.; 30; B.S. in engineering at Princeton University, 1926; New York State license; author of several technical papers; unusual background of underpinning, subway, and heavy construction work; formerly chief assistant to well known consulting engineer interested in experiments with earth pressures. Desires teaching position. C-9232.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 25; M.S. in structural engineering; 1 year experience in design of high dam; 1 year as instructor in civil engineering; experienced in steel and concrete design. Interested in teaching and research or in exploration and field work in mountain country. Available May 15. D-446.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 25; married; university graduate, 1927; 1 year with consulting sanitary engineers as draftsman and field engineer; 2½ years with structural steel fabricator as draftsman, estimator, and field engineer; 2½ years overhead power line design. Interested in teaching mathematics, surveying, and structural engineering. East or Middle West preferred. D-466.

YOUNG MAN; Jun. Am. Soc. C.E.; position as instructor desired; graduate civil engineer from leading technical university. No previous teaching experience but have had 7 years experience in surveying, drafting, and power plant construction. Capable of teaching mathematics, science, and civil engineering subjects. Location

in South preferred, but will consider any proposition. C-3182.

GRADUATE CIVIL ENGINEER; M. Am. Soc. C.E.; bachelor and professional degrees in engineering from prominent university; 20 years experience combining teaching and professional practice. Have been associated with leading engineering organizations and have had direct charge of design and construction of many important structures. Interested in an opening to teach structural engineering. A-2120.

STRUCTURAL ENGINEER; Jun. Am. Soc. C.E.; 31; single; instructor in civil engineering; graduate; licensed professional engineer, New York State; 1 year experience teaching civil engineering subjects; 9 years experience in structural design of commercial buildings, mill buildings, steam and hydro-electric power plants, floating derricks, cofferdams, and marine construction equipment. Location immaterial. Available now. B-5418.

RECENT BOOKS

New books of interest to Civil Engineers, recently donated by the publishers to the Engineering Societies Library, will be found listed here. A comprehensive statement regarding the service which the Library makes available to members is to be found on page 87 of the Year Book for 1932. The statements made regarding the books are taken from the books themselves and this Society is not responsible for them.

ADVANCED MECHANICS OF MATERIALS. By F. B. Seely. New York, John Wiley & Sons, 1932. 331 pp., illus., diagrs., charts, tables, 9 × 6 in., cloth, $5.

This textbook is intended for advanced undergraduate and first-year graduate students and continues the subject beyond the usual course in the strength of materials. It aims to show the limitations of the ordinary formulas, to consider the conditions under which the limitations are significant, and to extend the subject to more complex topics. It also endeavors to give a more comprehensive view of the fundamental concepts and methods used in stress analysis, and to acquaint the student with the principal sources of information upon the subject.

BEITRÄGE ZUR GESCHICHTE DER TECHNIK UND INDUSTRIE, Vol. 21. Ed. by C. Matschoss. Berlin, VDI-Verlag, 1931–1932. 188 pp., illus., diagrs., maps, tables, 12 × 8 in., cloth, 10.80 rm.

The twenty-first volume of this annual of engineering history contains a number of valuable papers, a review of articles published elsewhere, reports on existing ancient engineering works and on museums of industry, and a bibliography of historical publications of the year. Among the subjects of essays are: the discovery of magnetic induction; the development of electric heating during the nineteenth century; early knowledge of the magnetic compass; the development of reinforced concrete; and the history of hydraulic works in China.

HIGHWAY CURVES, 2 ed. By H. C. Ives. New York, John Wiley & Sons, 1931. 375 pp., diagrs., charts, maps, tables, 7 × 4 in., leather, $4.

A handbook on highway curves and earthwork for the use of highway engineers, with the necessary tables. The work follows the familiar plan of books on railroad location and earthwork and discusses simple and compound curves, widening and banking curves, earthwork problems, highway location, and the use and adjustment of instruments. The practice of various state highway commissions has been considered. The principal change in this edition is the inclusion of four new tables.

HYDRAULIC DIAGRAMS FOR CHANNELS IN EARTH, GIVING DISCHARGES, MEAN VELOCITIES, AND SILTING DATA FOR VARYING BED WIDTHS, DEPTHS, AND SLOPES, BASED ON GANGUILLET AND KUTTER'S FORMULA, 2 ed. By R. G. Kennedy. London, E. & F. N. Spon, Ltd.; New York, Spon & Chamberlain, 1931. 5 pp., charts, tables, 12 × 20 in., leather, $7.50.

These diagrams give by inspection the mean velocities and discharges of all ordinary earthen channels, including those from 1 to 12 ft. deep with slopes from 0.10 to 0.50 in 1,000. An additional table gives the data necessary to design the section and slope of channels so that they will have non-silting velocities. The diagrams are clearly printed and are accompanied by explanations. This edition has been revised.

LEISTUNGSVERSUCHE AN MISCHMASCHINEN. (*Mitteilungen des Forschungsinstituts für Maschinenwesen beim Baubetrieb, Heft* 1.) By G. Garbotz and O. Graf. Berlin, VDI-Verlag, 1931. 26 pp., illus., diagrs., charts, tables, 12 × 8 in., paper, 7.50 rm.

This pamphlet is the first of a series devoted to the results of the research work upon the uses of machinery in building construction which is carried on at the Berlin Technical University. In this first number are given the results of careful tests of concrete mixers made by eighteen German firms. The influence of mixing time, method of adding water, size of mixer drum, speed, and similar items upon the strength of the concrete have been investigated and conclusions reported.

MECHANISCHE KLÄRANLAGEN FÜR STADTE UND GEMEINDEN. By O. Mohr. München and Berlin, R. Oldenbourg, 1931. 40 pp., illus., diagrs., tables, 7 × 5 in., paper, 2 rm.

This little pamphlet is concerned with mechanical sewage purification plants for small communities. The principles underlying the planning of these plants and the precautions necessary in operating them are set forth in the briefest manner.

MEHRSTIELIGE RAHMEN, Bd. 1, Lief. 1, 3 ed. By A. Kleinlogel. Berlin, Wilhelm Ernst, 1931. 96 pp., diagrs., tables, 10 × 7 in., paper, 7.20 rm.

This work is intended to aid the structural designer by supplying him with formulas for the design of a variety of statically indeterminate frameworks which occur frequently in practice. The first volume covers fifteen common and seventeen special forms of rectangular framing. Formulas are given for determining all the data usually needed. The arrangement is unusually convenient for quick reference.

MOVEMENT OF PIERS DURING CONSTRUCTION OF MULTIPLE-SPAN REINFORCED CONCRETE ARCH BRIDGES. By W.M. Wilson. *Univ. Ill. Bul.—Eng. Experiment Station Bul.*, No. 234, Vol. 29, No. 10, Oct. 2, 1931. 30 pp.

Report on observations made under supervision of the author on five multiple-span bridges with piers resting either on rock or on piles driven into gravel; movement of piers, although small, produced an appreciable strain in the concrete; movements will not be excessive if the proper precautions are taken.

PLANNING FOR GOOD ACOUSTICS. By H. Bagenal and A. Wood. New York, E. P. Dutton & Co., 1931. 415 pp., illus., diagrs., charts, tables, 10 × 6 in., cloth, $6.75.

This treatise, the joint work of an architect and a physicist, is intended to give architects the acoustical information necessary to the proper design of modern buildings. It is a happy blend of the results of the experimental work of Professor Sabine and others with the knowledge gained from actual buildings in many countries. Such subjects as designing for musical requirements and for debate; the designing of theaters, dwellings, churches, and schools; the use of absorbents; sound transmission and soundproofing are discussed in a practical way.

PLANNING PROBLEMS OF TOWN, CITY, AND REGION. PAPERS AND DISCUSSIONS AT 23RD NAT. CONFERENCE ON CITY PLANNING, ROCHESTER, N.Y., JUNE 22 TO 24, 1931. Publ. for Conference, Wm. F. Fell Co., Phila., 1931 228 pp. illus; tables, diagrs. $3.

Topics discussed are: Is City Planning Effectively Controlling City Growth; Mass Transportation on City Streets; Master Plans and Official Maps; Zoning Administration; Edges of Zones; County Planning; Planning Programs for Small Cities. Eng. Soc. Lib. N.Y.

POPULATION CHARACTERISTICS BY CENSUS TRACTS, CLEVELAND, OHIO, 1930. By H. W. Green. Cleveland, Plain Dealer Publishing Co., 1931. 236 pp., charts, tables, maps, 12 × 9 in., cloth, $25.

This volume presents an elaborately detailed analysis of the population of Cleveland and its satellite cities, based upon the fifteenth decennial census of the United States. The distribution, age, sex, race, and nativity of the population; its citizenship, marital condition, literacy, employment, and characteristics; the size of families, ownership of homes, and other data concerning the economic and social condition of the inhabitants are given statistically and graphically for each census tract. The analysis is the most complete ever made of an urban community and provides basic data for city officials, public utilities, and business men interested in municipal affairs and business.

PRINCIPLES OF STRUCTURAL GEOLOGY. By C. M. Nevin. New York, John Wiley & Sons, 1931. 303 pp., illus., diagrs., charts, tables, 9 × 6 in., cloth, $3.50.

A text for beginners which aims to discuss, as simply as possible, the deformations of the earth. The presentation is clear and effective. Matters in dispute are not avoided, but attention is called to different views, and references that present them are given. The author endeavors to stimulate the ability to reason rather than to present the subject.

RAILROAD CONSTRUCTION, THEORY, AND PRACTICE. 9 ed. By W. L. Webb. New York, John Wiley & Sons, 1932. illus., diagrs., charts, tables, 8 × 5 in., leather, $6. Pt. 1, 699 pp.; Pt. 2, tables, 226 pp.

This text, dealing with all phases of construction from the preliminary survey to the completion of the road and provision of the rolling stock, has long been a favorite. The present edition is said to be very largely new, the author having incorporated in it the improved methods which have been devised in recent years. The necessary mathematical tables are included.

STANDARD CONSTRUCTION METHODS. 2 ed. By G. Underwood. New York and London, McGraw-Hill Book Co., 1931. 501 pp., illus., diagrs., charts, tables, 9 × 6 in., leather, $5.

A manual of recommended methods for all phases of practical work, intended for superintendents of construction and others responsible for getting work done quickly and efficiently. The methods described are those found most useful in ordinary work, and the descriptions are precise and clear. The new edition has been considerably enlarged and revised. Among the new subjects are forms for concrete floors, leaks in brick buildings, house moving, and towers and elevators for hoisting materials.

TREATISE ON PHOTO-ELASTICITY. By E. G. Coker and L. N. G. Filon. London, Cambridge University Press; New York, Macmillan Co., 1931. 720 pp., illus., diagrs., charts, tables, 11 × 7 in., cloth, $17.50.

This handsome volume, in which two leading authorities in the field of photo-elasticity present a comprehensive survey of the subject, will be indispensable to all students of photo-elasticity. The book opens with a presentation of the elements of the theory of physical optics and the theory of elasticity, after which an account of the historical development of photo-elasticity is given. The remainder of the book deals with various problems of scientific and practical engineering interest, such as the flexure of beams, holes, and cracks, stresses in planes involving straight and circular boundaries, and the testing of materials. A chapter is devoted to methods for determining the effects of load and strain on structures and machines. The illustrations in color are of unusual excellence, and a good bibliography is included.

ZUR BEHANDLUNG BAUSTATISCHER AUFGABEN ALS RANDWERTPROBLEME. By K. Schäfer. München u. Berlin, R. Oldenbourg, 1931. 77 pp., illus., diagrs., charts, tables, 10 × 7 in., paper, 5 rm.

This is a mathematical introduction to the statics of structures, intended for readers with some knowledge of statics. A uniform method for the solution of the problems that arise is presented, in which the differential equations and the boundary values that characterize them form the starting-point. This method gives great prominence to the influence line, which is shown to be a function of two variables, and gives results that can be readily used in practice. The necessary mathematical aids are included.

CURRENT PERIODICAL LITERATURE

Abstracts of Articles on Civil Engineering Subjects from Magazines in This Country and in Foreign Lands

Selected items from the current Civil Engineering Group of the Engineering Index Service, 29 West 39th Street, New York, N.Y. Every article indexed is on file in The Engineering Societies Library, one of the leading technical libraries of the world. Some 2,000 technical publications from 40 countries in 20 languages are received by the library and are read, abstracted, and indexed by trained engineers. With the information given in the items which follow, you may obtain the article from your own files, from your local library, or direct from the publisher. Photoprints will be supplied by this library at the cost of reproduction, 25 cents per page, or technical translations of the complete text may be obtained when necessary at cost.

BRIDGES

ARCHES, DESIGN. Comparisons of Arch Abutments, W. S. Wilson. *Civ. Eng.* (*London*), vol. 26, no. 307, Jan. 1932, pp. 30–31. Cost comparison of several abutment designs.

CANADA. Perly Inter-Provincial Highway Bridge. *Can. Engr.*, vol. 62, no. 6, Feb. 9, 1932, pp. 7–9 and 47. Description of the deck-type steel truss bridge, across the Ottawa River between Hawkesbury and Grenville, 2,228 ft. long; there is a roadway 24 ft. wide between curbs, and one sidewalk; the longest deck Warren girder span is 224 ft.

CONCRETE ARCH, AUSTRALIA. Construction of Grey Street Bridge, Brisbane, Australia, G. O. Boulton. *Civ. Eng.* (*London*), vol. 26, no. 306, Dec. 1931, pp. 55–58. Construction of bridge over Brisbane River comprising three 238-ft. reinforced concrete arch spans; unusual methods used for deep foundations; provision of "artificial islands"; combined use of open dredging and airlocks; submerged pier bases; concrete seals; caisson disease.

CONCRETE, ARKANSAS. Double Piers for Expansion in Rigid-Frame Bridge, F. C. Hart. *Eng. News-Rec.*, vol. 108, no. 6, Feb. 11, 1932, p. 207. Design and construction of reinforced-concrete bridge with a rigid-frame slab deck for crossing over Crooked Creek at Harrison, Ark.; the bridge consists of ten spans and has a length of 401 ft.; use of double elastic piers and elastic abutments to take care of temperature changes.

GERMANY. Cologne-Mulheim Suspension Bridge, K. Mohringer. *Civ. Eng.* (*London*), vol. 26, no. 307, Jan. 1932, pp. 33–40, 1 supp. plate. Design and construction of the largest suspension bridge in Europe, having a main span 1,033 ft. in length and two side spans each of 300 ft.; details of stiffening girders; weights and costs.

MASONRY ARCH, PHILADELPHIA. Brick Arch of 42-Ft. Span Built Without Falsework, *Eng. News-Rec.*, vol. 108, no. 4, Jan. 28, 1932, pp. 118–120. Design and construction of a 42-ft. span, arch bridge, at the Melrose Country Club near Philadelphia, having a rise of 6 ft., erected without falsework by cantilevering out from either abutment, and brickwork laid up on short centers suspended from the cantilever arms; bridge designed to carry a 4,000-lb. tractor running directly over spandrel walls.

PILES, WOODEN, PRESERVATION. Jacketed Creosoted Piles Repel Marine Borers, W. E. Wheat. *Eng. News-Rec.*, vol. 108, no. 6, Feb. 11, 1932, pp. 208–209. Casing of wood piling for Lillian Bridge trestle, 4,800 ft. long, built of untreated wood to bridge Perdido Bay in Florida.

RAILROAD, RECONSTRUCTION. Bridge Reconstruction on London Division, Canadian Pacific Railway. *Can. Ry. and Mar. World*, no. 408, Feb. 1932, pp. 55 and 70. Details of steel bridges reconstructed on London division; dismantling and construction operations.

STEEL TRUSS, AUSTRALIA. Combined Railway and Road Bridge at Grafton, N.S.W. *Commonwealth Engr.*, vol. 19, no. 6, Jan. 1, 1932, pp. 203–206. Design and construction of bridge comprising six main spans—five of 240 ft. each and one of 84 ft.; lift span of bascule type.

SUSPENSION, NEW YORK. County Is Building Two Suspension Bridges, W. G. Grove. *Roads and Streets*, vol. 75, no. 2, Feb. 1932, pp. 104–108. Features of proposed highway suspension bridges at Greenport, N.Y. and Sag Harbor, N.Y. having main arch spans close to 1,100 ft. and 700 ft.

TRESTLE. Maintenance of Ballast-Deck Trestles. *Ry. Eng. and Maintenance*, vol. 28, no. 2, Feb. 1932, pp. 97–100. Report on methods and workmanship designed to effect economies and avoid injury of treated wood; outline of maintenance and repair operations. Before Am. Ry. Bridge and Building Ass'n.

VIADUCTS, CONCRETE. Double-Deck Viaduct Built Over Cincinnati Terminal. *Eng. News-Rec.*, vol. 108, no. 6, Feb. 11, 1932, pp. 202–205. Design and construction of double-deck viaduct 3,500 ft. long, of reinforced-concrete and concrete-encased structural steel, crossing Mill Creek Valley at Cincinnati, comprising one reinforced concrete arch, 120-ft. span and 41-ft. rise, and similar arch of 109-ft. span and 62-ft. rise; difficult foundation work encountered; steel-pipe piles; heaving of concrete piles.

BUILDINGS

CONSTRUCTION. Testing Men and Materials for Rockefeller City, W. B. Todd. *Arch. Forum*, vol. 56, part 2, no. 2, Feb. 1932, pp. 199–204. Selection of general contractors; progress schedule developed to show, by dates and by floors, activity in each trade; selection of materials; budget of cost on office building; office partitions; exterior walls.

DESIGN. Deflections and Vibrations in High Buildings, L. J. Mensch. *Am. Concrete Inst.—Journal*, vol. 3, no. 6, Feb. 1932, pp. 387–404. Derivation of working rule for finding shears in columns of wind bracing system by moment-area method; relative stiffness of columns and girders; vibrations of tall buildings; examples; earthquake vibrations.

ELECTRIC WELDING. Arc-Welded Factory Building. *Gen. Bldg. Contractor*, vol. 3, no. 2, Feb. 1932, pp. 7–10. Construction of first American 100 per cent arc-welded steel frame building—new $1,000,000 tank shop, Building No. 14, at Pittsfield, Mass.; works of General Electric Company; pilasters covered with plates; all-steel framework erected during sub-zero weather; details of crane girder diaphragm and impost connections to exterior column.

FIREPROOFING. Fireproofing Steel-Framed Buildings with Cinder Concrete, J. A. Garrod. *Concrete*, vol. 60, no. 2, Feb. 1932, pp. 13 and 14. Methods employed in New York City; cost of work; location and erection of hoist; wire caging of beams, girders, and columns.

HISTORY. Wrecking Home Insurance Building Reveals Excellence of Old Metals, H. Penn. *Eng. News-Rec.*, vol. 108, no. 7, Feb. 18, 1932, pp. 247–248. First skeleton-frame building dismantled to make way for new Field Building in Chicago, was built of cast iron, wrought iron, and the first steel beams used in building in the United States.

STEEL, WELDED. Completely Welded Steel-work and Composite Frame *Contract Rec.*, vol. 46, no. 7, Feb. 17, 1932, pp. 145–150. Design and construction of six-story Canadian General Electric Company warehouse and office building, Montreal; floor-block system of floor design.

CONCRETE

COLD WEATHER. Winter Concreting—Methods and Details. *Contractors and Engrs. Monthly*, vol. 24, no. 2, Feb. 1932, pp. 23 and 24. Third of a series of articles dealing with the practical side of cold weather concrete construction; insulating of job from weather; heating enclosure; calcium chloride as accelerator; examples of winter construction.

COST. Costs and Prices of Ready-Mixed Concrete. *Gen. Bldg. Contractor*, vol. 3, no. 2, Feb. 1932, pp. 11–15. Manufacturing costs and selling prices of Bridgeport Concrete Company plant, Bridgeport, Conn.; labor rates, production costs, and trucking expense; material costs per yard of concrete; selling prices per cubic yard of concrete; relation of production, hauling, and material costs to selling prices.

PLANTS. Central Concrete Mixing Plant, Puget Sound Navy Yard, A. D. Hunter. *Eng. and Contracting*, vol. 71, no. 2, Feb. 1932, pp. 37 and 38. Main features and equipment of plant having a maximum capacity of 300 cu. yd. per 8-hour shift; aggregates brought in on barges; mixing concrete; gravel crushing; operating force.

RAILROAD TIES, CONCRETE. Concrete Permanent Way. *Ry. Engr.*, vol. 53, no. 625, Feb. 1932, pp. 68–71. Design and construction details of Wirth's spring-supported system and Pere Marquette system; test results of both systems; economic factors.

READY-MIXED. Problems of Ready-Mixed Concrete Industry Reviewed. *Eng. News-Rec.*, vol. 108, no. 6, Feb. 11, 1932, p. 219. Second annual convention shows active interest in technical problems; overproduction looms as an obstacle to profitable operation.

SLAG PLANTS. Five New Slag Plants Erected in 1931 Indicate Faith in Industry. *Pit and Quarry*, vol. 23, no. 8, Jan. 13, 1932, pp. 63–65. Features of recently opened slag plants in United States, utilizing by-products of blast furnaces.

TESTING. Tests of Concrete from Transit Mixer, S. C. Hollister. *Am. Concrete Inst.—Journal*, vol. 3, no. 6, Feb. 1932, pp. 405–417. Tests of variety of concrete mixtures made in 1, 2½, and 4-cu. yd. Jaeger truck-transit mixers; mixing time up to 90 min.; 22 runs were made, involving the making of over 800 cylinders; relation between strength of concrete and time of mix; relation of time of mix to degree of uniformity of concrete produced; relation between strength and water-cement ratio; minimum time of mix; effects of prolonged mixing.

UNITED STATES. Ready-Mixed-Concrete Industry Advances Rapidly and Improves Product. *Pit and Quarry*, vol. 23, no. 8, Jan. 13, 1932, pp. 59–62 and 77. Annual review of progress; many new plants spring up near larger centers of population during the year.

WORKABILITY. Studies of Workability of Concrete, T. C. Powers. *Am. Concrete Inst.—Journal*, vol. 3, no. 6, Feb. 1932, pp. 419–448. Progress report on study of the workability of concrete mixtures by means of a new test, called "remodelling test," which measures relative effort required to change the mass of concrete from one definite shape to another by means of jigging; effects of amount of paste and percentage of sand; effect of consistency of paste on mobility of concrete; effect of admixtures on mobility; gap grading and "sized" materials.

CONSTRUCTION INDUSTRY

COSTS. Unit Bid Summary. *West. Construction News*, vol. 6, no. 3, Feb. 10, 1932, pp. 96, 98, and 126. Unit costs bid on railroad construction, street and road work, and bridge construction, in California, Washington, and Arizona.

UNITED STATES. Facts and Figures of Current Construction Progress. *Eng. News-Rec.*, vol. 108, no. 5, Feb. 4, 1932, pp. 159–166. Annual review of construction industry in the United States; construction volume declined despite increased public works; costs are also lower; construction financing hampered; production of construction materials less than in previous years; skilled and common construction wages; estimates indicate decrease for 1932; public construction budgets of eleven cities.

DAMS

BUTTRESS. New form of Dam Proposed for Narrow Rock Canyons, C. H. Howell. *Eng. News-Rec.*, vol. 108, no. 6, Feb. 11, 1932, pp. 214–216. Features of original thrust-buttress type of dam, with buttresses abutting in rock of either canyon wall, with vertical upstream surface of L-shaped alignment; this type is claimed to be proof against overturning, sliding, and uplift effects and is from 40 to 60 per cent smaller than a gravity structure; buttresses act as columns.

CALIFORNIA. Pine Canyon Dam for Pasadena Water Supply. *West. Construction News*, vol. 7, no. 3, Feb. 10, 1932, pp. 81–83. Main features of concrete gravity dam 245 ft. high above stream bed, and 750 ft. long on crest, estimated to cost $5,300,000, including materials.

CLEANING. Cleaning Impounding Reservoir with Suction Dredge. *Am. City*, vol. 66, no. 2, Feb. 1932, pp. 69 and 70. Division of Water, Department of Public Works, Newark, N.J., uses "vest pocket" unit which can be knocked down for cross-country hauling.

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COLORADO. Material Handling on Colorado Dam, J. C. Coyle. *Contractors and Engrs. Monthly*, vol. 24, no. 2, Feb. 1932, pp. 28–29 and 32. Construction of Eleven-Mile Canyon Dam for the Water Department of Denver, Colo.; the dam, which is of concrete gravity type, is 54 ft. thick at the base and 15 ft. at the top, its height is 147 ft. and length 445 ft.; sand from quarried material; excavation; handling aggregate to mixer.

CONCRETE. Design and Construction of Service Reservoirs, M. R. Barnett. *Civ. Eng. (London)*, vol. 26, no. 307, Jan. 1932, pp. 23–26. Review of the British practice of design and construction of open and covered water distribution reservoirs; recent examples.

DESIGN. Stability of Dams Increased by More Economical Use of Materials, C. V. Davis. *Eng. News-Rec.*, vol. 108, no. 6, Feb. 11, 1932, pp. 210–214. Progressive changes from solid gravity dam to buttress type show increasing stability with decrease of indeterminate factors; comparison of types; combined arch and brace overcomes objectionable features; advantages of wider buttress spacing; comparison of structural and economic characteristics.

EROSION. Erosion Below Conowingo Dam Proves Value of Model Tests, L. L. N. Reeve. *Eng. News-Rec.*, vol. 108, no. 4, Jan. 28, 1932, pp. 127–130. Design based on results of tests made in the Worcester laboratory is found effective in minimizing erosion and in insuring that it occurs only at a safe distance below the dam.

GRAVITY MASONRY, RAISING. Aswan Dam. *Civ. Eng. (London)*, vol. 26, no. 305, Nov. 1931, pp. 36–43. Account of the work being carried out on behalf of the Egyptian government for a second heightening of the dam from R. L. 114 m. to R. L. 123 m., involving an extension of abutments, west and east, by approximately 66 and 81 m., respectively; buttresses added between sluices; vertical asphalt seals are being introduced.

INDIA. Lloyd Barrage at Sukkur. *Engineering*, vol. 133, no. 3446, Jan. 29, 1932, pp. 128–130 and 132. Completed dam consists of two sets of reinforced concrete arches, forming 66 double 60-ft. spans, separated by 58 ordinary piers, each 10 ft. wide, and seven abutment piers, each 25 ft. wide.

OUTLETS. Control of Water in Large Volumes and Under High Velocity of Discharge Through Dams and Pressure Tunnels, E. B. Ball. *Water and Water Eng.*, vol. 34, no. 399, Jan. 20, 1932, pp. 27–30. Design and operating conditions of control apparatus for large volumes of water and high heads. Before Inst. Mech. Engrs.

UNITED STATES. Dams—High, Large, and Unusual—I. P. I. Taylor. *Reclamation Era*, vol. 23, no. 2, Feb. 1932, pp. 28–32. Main features of the most remarkable dams, built in the United States, of masonry, earth, or rock-fill; list of high dams in United States built between 1910 and 1932.

WEIRS, APRONS. Depressed Apron Checks Erosion Below Hamilton Dam, C. H. Eiffert. *Eng. News-Rec.*, vol. 108, no. 4, Jan. 28, 1932, pp. 130–132. Description of concrete weir, about 5 ft. high, built by Miami Conservancy District, Dayton, Ohio; flexible mat on river bed proves unsatisfactory; energy dissipation now accomplished in pool over depressed apron; seepage under dam.

FLOW OF FLUIDS

MEASUREMENT. Coefficients of Discharge of Commercial Metering Elements in 3-in. Pipe, S. R. Beitler. *Water and Water Eng.*, vol. 34, no. 399, Jan. 20, 1932, pp. 10 and 11. Ohio State Engineering Experiment Station investigation of the characteristics of the flow of fluids through various commercial metering elements in 3- and 6-in. pipe lines, including sets of square-edge orifices, (concentric, eccentric, and segmental) flow nozzles, and venturi tubes; water and superheated steam were used as calibrating fluids.

ORIFICES. Measurement of Fluid Flow—Tests on Flow of Water Through Orifice Plates, F. V. A. Engel. *Civ. Eng. (London)*, vol. 26, no. 306, Dec. 1931, pp. 21–27. Results of experiments carried out by the author at City and Guilds Engineering College (Imperial College) London; discharge coefficients for orifices; layout of test plant; impact pressure. Bibliography.

FOUNDATIONS

CONCRETE. Concrete Foundations Treated as Engineering Problem, R. H. Vose. *Concrete*, vol. 60, no. 2, Feb. 1932, pp. 25 and 26. First of a series of seven articles dealing with inspection, testing, and field supervision of concrete work; types of foundations and their relation to the soil; pile foundations; effect of freezing on foundations.

PILES, CONCRETE. Piles 93 Ft. Long at Renfrew, W. L. Scott. *Concrete and Constr. Eng.*, vol. 27, no. 2, Feb. 1932, pp. 95–100. Method of driving concrete piles, up to 93 ft. in length, for foundations of viaduct carrying length of new Renfrew Road and a 75-ft.-span girder bridge over Maxwell Road, Glasgow; driving record for 17-in. octagonal pile, 93 ft. long.

RETAINING WALLS. Unusual Retaining Wall Design. *Surveyor*, vol. 81, no. 2089, Feb. 5, 1932, p. 193. Design and construction of cellular type of concrete and masonry retaining wall with maximum height of 24 ft.

SOILS, TESTING. Method of Determining Compaction and Shrinkage of Soil. *Eng. and Contracting*, vol. 71, no. 2, Feb. 1932, pp. 63 and 64. Abstract of report by the joint committee of the Highway Research Board and the American Road Builders Ass'n. on the California State Highway Commission's method of determining the degree of compaction of soil embankments and excavated slopes; method of sampling; testing machine and field methods of compacting; determining compaction and shrinkage.

HYDRAULIC ENGINEERING

RIVER HYDRAULICS. Some Interesting Aspects of River Hydraulics, G. S. Knapp. *Kansas Eng. Soc.—Year Book*, 1931, pp. 20–23, and (discussion) 24. Discussion of hydraulic formulas, particularly Kutter's formula for the flow of water in open channels, and such formulas as Dunn's and Talbot's for bridge openings, results of the application of these formulas without general knowledge of peculiar characteristics of flow of Kansas streams.

INLAND WATERWAYS

GATES. Welland Canal Gate Lifter Subjected to 500-Ton Test, A. F. Case. *Eng. News-Rec.*, vol. 108, no. 3, Jan. 21, 1932, pp. 86 and 87. Floating hoist designed to operate in limited lock chamber found stable under heaviest pontoon lift ever attempted.

PANAMA CANAL. The Canal Situation, J. F. Stevens. *Baltimore Engr.*, vol. 6, no. 11, Feb. 1932, pp. 4–13 and 22. Reminiscences and comments by the first chief engineer charged with the construction of the Panama Canal; feasibility and necessity of second inter-oceanic canal through Lake Nicaragua is discussed.

SHOALS. Maintaining River Channels by Shoal Dredging, J. J. Sullivan. *Eng. News-Rec.*, vol. 108, no. 2, Jan. 14, 1932, pp. 50–53. Location and occurrence of shoals; mobilizing for dredging parallel cut system employed; dustpan suction head preferred; locating dredge cuts, system of dredging by which series of parallel cuts are made upstream through shoal; utilizing aid of current.

IRRIGATION

MAINTENANCE. Canal-Bank Maintenance by Disking on Imperial Irrigation District, M. J. Dowd. *Eng. News-Rec.*, vol. 108, no. 6, Feb. 11, 1932, pp. 206–207. Development of machine operations for the removal of plant growth on ditch banks; tractor-drawn disk supplants hand-cutting of brush and bamboo on 2,600 miles of ditch; cost cut in half and results more effective; unit completes 25 miles per month.

UNITED STATES. Methods of Alleviating Water Shortage on Irrigation Projects, G. D. Clyde. *Agric. Eng.*, vol. 13, no. 2, Feb. 1932, pp. 37–41. Watershed management; forecasting water supplies; snow-cover-runoff relations; supplemental water supplies by pumping; replenishing groundwater supplies artificially; distribution and application of water; fall and winter irrigation.

MATERIALS TESTING

BUILDING STONE, TESTING. Building Stone and Slate. *Am. Soc. Testing Materials—Proc.*, vol. 31, part 1 of meeting, June 22–26, 1931, pp. 829–847. Tentative methods of testing and definitions.

CONCRETE. Hot Cement Has No Ill Effects in Concrete Paving. *Eng. News-Rec.*, vol. 108, no. 7, Feb. 18, 1932, pp. 244–245. Tests made on alternate sections of Indiana concrete road paved with hot cement and cold cement indicate no damaging effects from high cement temperatures.

METALS, COHESION. Rupture and Elasticity of Metals, R. D. Archibald. *Mech. World*, vol. 91, no. 2350, Jan. 15, 1932, pp. 51–52. Electromagnetic analogy explaining phenomena of elasticity.

SPECIFICATIONS. Cement, Lime, Gypsum, Concrete, and Clay Products. *Am. Soc. Testing Materials—Tentative Standards*, 1931, pp. 220–323. Tentative specifications, methods of testing, and definitions.

STEEL. On Mechanism of Break-Down of Steel, M. Kuroda. *Inst. Phys. and Chem. Research—Sci. Papers*, vol. 17, no. 340, Dec. 1931, pp. 111–123, 10 supp. plates. Experimental investigation of appearances of discontinuities in structure of soft steel at various temperatures from room temperature to 300 deg. cent.; relationship of yield point with Lueders' line. (In English.)

STRESSES, MEASUREMENT. New Instrument Measures Stresses in Dams, A. Floris. *West. Construction News*, vol. 7, no. 2, Jan. 25, 1932, pp. 51 and 52. Strain measurements of concrete dams by means of various mechanical and electrical instruments; telemeters; ultra-micrometers; absorption micrometer; detail of cartridge of oscillating condenser. Bibliography.

WELDS. Non-Destructive Testing of Welds. *Can. Engr.*, vol. 62, no. 5, Feb. 2, 1932, pp. 17 and 18 and 52. Brief review of methods of testing welded products in shop or field; air-pressure test; testing welded pressure piping with stethoscope; reheating of welds.

MUNICIPAL ENGINEERING

GREAT BRITAIN. Municipal Engineering in 1931. *Surveyor*, vol. 81, no. 2088, Jan. 29, 1932, pp. 93–132. Review of progress with special reference to highways and highway bridges, sewage disposal and drainage, water works, and housing.

PORTS AND MARITIME STRUCTURES

CALCUTTA, INDIA. Port of Calcutta. *Engineering*, vol. 133, no. 3445, Jan. 22, 1932, pp. 110–111. Abstracts of papers before the Institution of Civil Engineers as follows: Calcutta Port Extensions, W. C. Ash; Construction and Sinking of Monoliths at King George's Dock, Calcutta, J. D. Pearson.

CARGO HANDLING. Pneumatic Cargo Handling in Ports, A. Jordahl. *Materials Handling and Distribution*, vol. 7, no. 4, Jan. 1932, pp. 24 and 25. List of units comprising pneumatic vacuum unloading plant; for the unloading, by air, of bulk materials in ports, any one of the following three distinct types may be used: floating pneumatic conveyor, traveling unloading plant, or stationary system.

CHINA. Port Development in North China, T. Lien-en. *Ass'n Chinese and Am. Engrs.—Journal*, vol. 12, no. 11, Nov. 1931, pp. 36–45. Study of port development at Hulutao, including 5,000 ft. of breakwater; 3,700 ft. of quay-wall, 300 acres of sheltered anchorage, 3,600,000 cu. yd. of dredging, and 150 acres of reclamation; harbor when completed will be capable of handling about six million tons of freight a year; berthing wall will measure 8,800 ft. in length, accommodating 16 vessels of average length of 400 ft.

CORROSION. Deterioration of Structures in Sea-Water. *Iron and Coal Trades Rev.*, vol. 144, no. 3331, Jan. 1, 1932, p. 3. Previously indexed from various sources.

LOS ANGELES. West Basin Terminal, Los Angeles Harbor, G. F. Nicholson. *West. Construction News*, vol. 7, no. 3, Feb. 10, 1932, pp. 85–86. Main features of $1,250,000 project consisting of two-berth wharf and transit shed, railroad tracks, roadways, and storage yard especially designed for handling pipe and steel as well as general cargo; transit shed designed for passenger accommodations.

RAILROADS, STATIONS, AND TERMINALS

MILAN, ITALY. New Central Railway Station at Milan. *Engineer*, vol. 153, no. 3967, Jan. 22, 1932, pp. 92–94 and 100. Structure comprises station building and track, raised 25 ft. above street level; area for platforms and lines is 5,900 ft. long and 660 ft. wide; platform roof, 1,125 ft. long, consists of five glass-covered arches, center arch having a clear span of 238 ft.; half of roof is covered with wired glass panels set in special frames without putty; construction details.

ROADS AND STREETS

AFRICA. British-African Road Development. *Roads and Road Construction*, vol. 10, no. 109, Jan. 1, 1932, pp. 8–11. Statistical review of recent developments in Basutoland, Bechuanaland Protectorate, Gambia, Gold Coast (including British Togoland), Kenya Colony and Protectorate, Mauritius, Nigeria; Northern Rhodesia, Nyasal and Protectorate; Sierra Leone, Sumalil and Protectorate, South West Africa; Southern Rhodesia, Swaziland, Tanganyika Territory, Uganda Protectorate, Union of South Africa, Zanzibar.

ASPHALT. Machine Finishing on Two-Course Asphalt Resurfacing. *Contractors and Engrs. Monthly*, vol. 24, no. 2, Feb. 1932, pp. 45–49. Outline of the methods developed for the Michigan State Highway Department for placing permanent type of surface on existing gravel roads which would be flexible enough to meet variable climatic conditions; work redesigned to fit machine operation; aggregate production; paving operations; spreading binder and sheet top; handling traffic; labor organization; production schedule.

BITUMINOUS. Mixed-in-Place Road Construction in East. *Am. City*, vol. 66, no. 2, Feb. 1932, pp. 85–90. Results of experimental work in South Carolina, Massachusetts, and Vermont.

BRICK. Metal Base Experimental Road Fails Under Test. *Concrete*, vol. 60, no. 2, Feb. 1932, pp. 15–16. 150-ft. section in Sangamon County, Ill., removed and replaced with concrete base; details of metal-base construction; impossibility of supporting block-type surface on flexible base.

CONCRETE. Mixing and Agitating Concrete for Roads. *Can. Engr.*, vol. 62, no. 5, Feb. 2, 1932, pp. 9 and 10 and 52. Construction of 80,000 sq. yd. of cement concrete pavements in city of Vancouver, B.C.; use of central mixing plant and subsequent mixing or agitating of concrete in trucks that carried batches to subgrade.

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DESIGN. Design of Concrete Pavements. W. E. Barker. *West. Soc. Engr.—Journal*, vol. 36, no. 6, Dec. 1931, pp. 355–362. Consideration of the following basic points: width, thickness, joints and joint spacing, and reinforcement.

FROST HEAVING. New Theory of Frost Heaving. A. C. Benkelman and F. R. Olmstead. *Am. Highways*, vol. 11, no. 1, Jan. 1932, pp. 13–16. Michigan State Highway Department investigation of effects of freezing upon soils, in laboratory; specially prepared specimens in glass tubes were frozen at a uniform rate of cooling; freezing of coarse sands in glass tubes; theory substantiated by field observations.

GREAT BRITAIN. Road Investigations. *Engineering*, vol. 133, no. 3446, Jan. 29, 1932, pp. 133–134; see also *Quarry and Roadmaking*, vol. 37, no. 420, Feb. 1932, pp. 49–51. Review of report by Technical Advisory Committee on Experimental Work, appointed by Minister of Transport, published by H. M. Stationery Office, Lond., price 5s. 6d. net; discussion of concrete roads constitutes major portion, main regard being paid to its use as foundation; bituminous surfaces; resistance of surfaces to skidding; vehicle impact.

MATERIALS. Review of Papers Read at Association of Asphalt Paving Technologists. *Rock Products*, vol. 35, no. 2, Jan. 30, 1932, p. 93. Determining surface area; test for designing cold road mixtures; method of operation of equipment to test cold road mixes.

UNITED STATES. Developments in Highway Construction and Highway Materials, P. J. Freeman. *Rock Products*, vol. 35, no. 1, Jan. 9, 1932, pp. 68–69. Brief notes on the durability of concrete; crushed gravel; low-cost roads; high early strength cements; hot cement; coal bituminous mixtures; emulsified bituminous materials; tar specifications.

SEWERAGE AND SEWAGE DISPOSAL

CHLORINATION. Chlorination of Sewage, C. K. Calvert. *Indus. and Eng. Chem.*, vol. 24, no. 1, Jan. 1932, pp. 92–94. Treatment of Indianapolis sewage; liquid-chlorine delivery devices; efficiency of application of chlorine; activated-sludge plant.

CONSTRUCTION. Sewer Construction on Temporary Trestle Through Marsh, T. F. Bowe. *Eng. News-Rec.*, vol. 108, no. 7, Feb. 18, 1932, p. 246. One section of 24-in. cast-iron outfall with lead joints for Hudson, N.Y., assembled in 244-ft. length in cable slings and lowered as unit.

DEARBORN, MICH. Chemical Treatment Adopted for Dearborn's Sewage. *Eng. News-Rec.*, vol. 108, no. 7, Feb. 18, 1932, pp. 252–253. Precipitation and sludge dewatering aided by pulped paper effluent filtered through thin bed of magnetite; lifted by magnet for washing.

DESIGN. Camouflaging Sewage Treatment Plants, F. N. Daniels and C. J. Capen, Jr. *Sewage Works, Journal*, vol. 4, no. 1, Jan. 1932 pp. 91–100. Methods of camouflaging sewage disposal plants, under varying conditions of location and site, in State of New Jersey, including communities of Totowa, West Paterson, Hawthorne, North Wildwood, Haddon Heights, Spring Lake, Long Branch, and Deal.

EXPLOSIONS. Explosions in Sewers and Sewer Maintenance, W. T. Knowlton. *Sewage Works, Journal*, vol. 4, no. 1, Jan. 1932, pp. 114–118. Causes of sewer explosions; combustible gas indicator; survey of gas conditions in sewer manholes; leakage of illuminating gas into sewers; odors; London explosions.

MAINTENANCE AND REPAIR. Sewer Maintenance in Philadelphia, A. H. Mainwaring. *Sewage Works, Journal*, vol. 4, no. 1, Jan. 1932, pp. 119–124 and discussion 131–134. Outline of methods developed for maintenance of 2,000 miles of sewers on area of 130 sq. miles; organization; office work; gang work; contract work; casting work; choked inlets; castings.

MANHOLES. Round Table. *Mun. Sanitation*, vol. 3, no. 2, Feb. 1932, pp. 76–79. Practical discussion of equipment and methods used in dewatering manholes and clogged sewers.

ODOR CONTROL. Indole and Skatole in Sewage, W. Rudolfs and N. S. Chamberlain. *Indus. and Eng. Chem.*, vol. 24, no. 1, Jan. 1932, p 111. Nitrogenous degradation products, indole and skatole, which produce penetrating odors when diffused into the air, are present in raw sewage, and to a less extent in settled sewage. Only traces of them are present in sewage treated by sprinkling filters or by activated-sludge process.

OPERATION. Operation of Typical Sewage Disposal Plants in California, E. A. Reinke. *Sewage Works Journal*, vol. 4, no. 1, Jan. 1932, pp. 83–90. Results of the survey of existing plants conducted by the California State Department of Public Health during 1930 and 1931; comments on operation; bar screens; Imhoff tanks; separate sludge digestion; siphons; sprinkling filters; secondary settling tanks; activated-sludge plants; sand filters; sewage ponds; chlorination.

SLUDGE. Sludge Digestion as Applied at Chorley Sewage Works, J. Slater. *Surveyor*, vol. 81, no. 2089, Feb. 5, 1932, p. 200. Review of practice of sludge digestion without artificial heating and no gas collection; starting digestion process; drawing off separated water; mixing of sludge.

TANKS. DESIGN. Incidence of Depth in Detritus Tank Design, A. S. Lowe. *Civ. Eng. (London)*, vol. 26, no. 306, Dec. 1931, pp. 28–29. Settling rates of mineral matter in sewage; settling rates of particles at different velocities of flow; path followed by particles.

TREATMENT. Trends in Sewage Treatment in 1931, F. W. Mohlman. *Water Works and Sewerage*, vol. 79, no. 2, Feb. 1932, pp. 33–37. Status of sludge digestion; disposal of digested sludge; mechanical dewatering of sludge; chemical processes; activated sludge; sprinkling filters.

VENTILATION. Sewer Ventilation, Why and How? D. Donaldson. *Sewage Works Journal*, vol. 4, no. 1, Jan. 1932, pp. 108–113. Summary of reasons for the ventilation of sewers; samples of British regulation for prevention of accidents in sewers.

STREET CLEANING AND REFUSE DISPOSAL

RAILROADS, SNOW REMOVAL. Winters Severe in Maine. *Ry. Eng. and Maintenance*, vol. 28, no. 2, Feb. 1932, pp. 94–96. Review of operating conditions and snow fighting equipment.

REFUSE DISPOSAL, Great Britain. Collection and Disposal of House Refuse, S. C. Baggott. *Inst. Mun. and County Engrs.—Journal*, vol. 58, no. 16, Feb. 2, 1932, pp. 1247–1254, and discussion 1254–1263. General discussion of ordinances, methods, and costs, with special reference to British municipal practice. Before Pub. Works, Roads, and Transport Congress.

STRUCTURAL ENGINEERING

COLUMNS, STEEL, FOOTINGS. Anchoring Billets and Columns by Welding, A. S. Coombs. *Eng. News-Rec.*, vol. 108, no. 6, Feb. 11, 1932, pp. 223 and 224. Details of welded anchorage construction designed to meet effect of upward water pressure on the basement floor of the extension to the commercial steam-heating plant at Kneeland St., Boston, for the Edison Electric Illuminating Company.

RETAINING WALLS. EARTH PRESSURE. Internal Movement of Sand, F. Takabeva. *Engineering*, vol. 133, no. 3447, Feb. 5, 1932, pp. 148–149. Particulars of Japanese experimental work carried out at Hokkaido Imperial University, Sapporo; investigations of the effect of sinking part of the bed on which the sand was resting, turning of retaining wall and its base, and forward movement of wall in vertical position; sand used was river sand, dried and sifted to grain size of 0.86 mm. and mixed with unspecified proportion of finer grain.

STATICALLY INDETERMINATE STRUCTURES. Structural Analysis Based on Moment Distribution Method of Cross, T. D. Mylrea. *Envrs. Soc. Western Pa.—Proc.*, vol. 47, no. 9, Nov. 1931, pp. 405–438. Application of the method developed by Hardy Cross to analyses of stresses, continuous beams, rigid frames, and similar structures.

WALLS, CONCRETE. Structural Performance of Concrete Masonry Walls, F. E. Richart. *Am. Concrete Inst.—Journal*, vol. 3, no. 6, Feb. 1932, pp. 383–385. Specifications as to materials, workmanship, and design of masonry walls; building code requirements; tests on stability of concrete masonry walls; effect of strength of unit on wall strengths; design of units, deformations of axially loaded walls; deformations in units and in mortar joints; behavior of composite walls; walls under eccentric loading; deformations of thin-wall units; factor of safety of walls.

SURVEYING

AERIAL. Methods of Eliminating Ground Surveying for Control in Aerial Photographic Mapping, E. Church. *College Applied Science—Syracuse Univ.*, 64 pp. Fifth of a series of papers on methods for eliminating ground control either entirely or partially; elimination of all ground control after first pair of photographs in series having 75 per cent or 60 per cent longitudinal overlap; control by elevations only; control by horizontal positions only.

TUNNELS

CALIFORNIA. Trans-Bay Bridge Tunnel Unit to Be World's Largest Cross-Section Bore, C. H. Purcell. *Calif. Highways and Pub. Works*, Jan. 1932, pp. 6, 7, and 17. Main features of tunnel unit of San Francisco-Oakland Bay Bridge project, 70 ft. wide, 50 ft. high, 500 ft. long.

RAILROAD, WIDENING. Bad Ground and Heavy Train Movements Complicate Enlargement of Single-Track Tunnel. *Construction Methods*, vol. 14, no. 2, Feb. 1932, pp. 34–37. Double-tracking of single-track tunnel, 1,894 ft. long, at Stretchers Neck, W.Va., on Chesapeake & Ohio railway line; blasting of old tunnel lining.

TELEPHONE CABLES, RIVER CROSSINGS. Some Unusual Conduit Tunnels, S. A. Haviland. *Bell Telephone Quarterly*, vol. 11, no. 1, Jan. 1932, pp. 27–33, 1 supp. plate. Tunnels constructed under Harlem River in New York City; limitations of submarine cable; construction methods.

WATER PIPE LINES

CLEANING. Pipe Pit Cuts Cleaning Costs, A. T. Clark. *Am. City*, vol. 66, no. 1, Jan. 1932, pp. 85–86. Layout of pipe cleaner pit for 16-in. main developed for Pocantico division of New Rochelle Water Company; bell-and-spigot pipe with 8-ft. center section connected with victaulic joints; cash value of cleaning.

RAILROAD CROSSINGS. Old Railroad Culvert Used for Water Pipe Crossing, A. E. Bennett. *Eng. News-Rec.*, vol. 108, no. 6, Feb. 11, 1932, p. 223. Crossing of 320-ft. wide right of-way of C. M. St. P. & P. Ry., in Aberdeen, S.D., by new 12-in. cast-iron pipe line, was made economically by pulling the pipe through an old 30-in. cast-iron culvert, by means of hand winch and steel cable.

WATER RESOURCES

UGANDA, AFRICA. Uganda, E. J. Wayland. *Min. Mag.*, vol. 46, no. 1, Jan. 1932, pp. 9–18. British protectorate in east central Africa, northwest of Lake Victoria; outline of geology, with special reference to water supply.

WATER TREATMENT

HARTFORD, CONN. Water Filtration at Hartford, W. A. Gentner. *New England Water Works Ass'n—Journal*, vol. 45, no. 4, Dec. 1931, pp. 360–371. Design and operation of plant serving a population of 212,000; slow sand-filtration; rate of filtration; summary of filter runs; raking; sand washing; re-sanding; operating costs; sand handling costs.

MANGANESE REMOVAL. Various Water Samples and Plants Were Examined to Learn Effects of Manganese in Filters, L. V. Carpenter. *Water Works Eng.*, vol. 85, no. 3, Feb. 10, 1932, pp. 136–138. Occurrence of manganese in water supplies of Northern West Virginia is due to drainage from bituminous coal mines; analysis of mine waste waters; effect of manganese on filter plants; examination of filter sands; effects of operation; orthotolidine reaction; effects on filtration. Bibliography. Before West Virginia Conference on Water Purification.

METALLIC. Sterilization of Water by Catadyn Silver, E. V. Suckling. *Water and Water Eng.*, vol. 34, no. 399, Jan. 20, 1932, pp. 15–18. Character of water; rate of filtration and reaction time; retained bactericidal power; factors impeding sterilization by silver; after-growth; algicidal action of catadyn silver; physical and chemical characters of treated water; minor applications; life of catadyn sand.

OPERATION. Operating Experiences at Cambridge, Mass., Filtration Plant, M. C. Whipple and F. E. Smith. *New England Water Works Ass'n—Journal*, vol. 45, no. 4, Dec. 1931, pp. 321–332. Review of operating experiences since 1927; principal features of works of nominal capacity of 14 m.g.d.; raw water sources; coagulation; chlorination; corrective treatment; cost of operation; additions to works.

PROVIDENCE, R.I. Five Years' Water Purification at Providence, E. L. Bean. *New England Water Works Ass'n—Journal*, vol. 45, no. 4, Dec. 1931, pp. 334–350. Raw water characteristics; changes in carbon dioxide, hydrogen ion concentration, and in dissolved oxygen during influent aeration; use of alum, lime, and chlorine; analysis of filter sand; effluent aeration; correction of hydrogen-ion concentration of aerated filter effluent by hydrated lime; summary of bacterial analyses.

TASTE AND ODOR REMOVAL. Activated Carbon Removes Tastes and Odors from Saginaw Water, A. Eckert. *Am. City*, vol. 66, no. 2, Feb. 1932, pp. 90–91. Applying powdered activated carbon; effects of activated carbon; amount of carbon needed varies with different waters; problems encountered. Before Sixth Annual Mich. Conference on Water Purification.

UNITED STATES. Development of Water Purification in New England, R. S. Weston. *New England Water Works Ass'n—Journal*, vol. 45, no. 4, Dec. 1931, pp. 315–320. Notes on evolution of water treatment by filtration alone or by filtration combined with aeration, chemical treatment or both.

WATER WORKS ENGINEERING

DESIGN. Small Water and Sewerage Schemes, H. C. Adams. *Surveyor*, vol. 81, no. 2087, Jan. 22, 1932, pp. 75–77. Pipe dimensions for small water supplies; misapplied formulas; differences in consumption; house requirements; tracing of leaks; doubtful economies; specifications.

DRILLING. Modern Practice in Artesian Well Drilling. *Civ. Eng. (London)*, vol. 26, no. 307, Jan. 1932, p. 32. Review of recent improvements in British practice.

WELLS, DEEP. Deep Well Water Services, J. S. Gander. *Steam Engr.*, vol. 1, no. 5, Feb. 1932, pp. 191–194. Brief outline of geological aspects; general definition of terms; characteristics of wells and pumping equipment of London basin; water well pumping; water analysis.